# JOURNAL OF THE INSTITUTION OF CIVIL ENGINEERS.

No. 8. 1939–40.
OCTOBER 1940.

## ORDINARY MEETING.

7 May 1940.

SIR CLEMENT DANIEL MAGGS HINDLEY, K.C.I.E., M.A., President, in the Chair.

It was resolved—That Messrs. F. H. Brunt, Robert Chalmers, D. C. Farquharson, A. S. Grunspan, R. W. Mountain, H. C. Ritchie, P. J. H. Unna, and J. S. Wilson be appointed to act as Scrutineers, in accordance with the By-laws, of the ballot for the election of the Council for the year 1940–41.

The Scrutineers reported that the following had been duly elected as—

*Members.*

HUGH EYRE CAMPBELL BEAVER.
JOHN LUCIAN SAVAGE.

*Associate Members.*

FREDERICK JOHN STRAFFORD BEST, Stud. Inst. C.E.
THOMAS MARSHALL CARTLEDGE, B.Sc. (Eng.) (*Lond.*), Ph.D. (Eng.) (*Lond.*), Stud. Inst. C.E.
HAROLD SAMUEL CARTMELL, Stud. Inst. C.E.
ROBERT FRANCIS DOUGLAS, B.Eng. (*Liverpool*), Stud. Inst. C.E.
LESLIE NORMAN FRASER, M.Eng. (*Liverpool*), Stud. Inst. C.E.
DAVID ARTHUR HOPKINS, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
DAVID MONCRIEFF REID JOHNSTONE, B.Sc. (*Edin.*), Stud. Inst. C.E.
ROYSTON JONES, M.Sc. Tech. (*Manchester*), Stud. Inst. C.E.
JOHN WILLIAM KEY, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
DANIEL LAMPERT, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
LINCOLN HARRY MARTIN, Stud. Inst. C.E.
JOHN BALFOUR ROSS, B.Sc. (*Edin.*), Stud. Inst. C.E.
STANTON JAMES DINGLE RUSSELL.
IAN HOGG STEWART, Stud. Inst. C.E.
EDGAR BURKE WILSON, B.Sc. (*Belfast*), Stud. Inst. C.E.
WILLIAM ARTHUR WOOTTON, Stud. Inst. C.E.

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

Paper No. 5233.

# "Cliff-Stabilization Works in London Clay."

By Jack Duvivier, B.Sc. (Eng.), M. Inst. C.E.

TABLE OF CONTENTS.



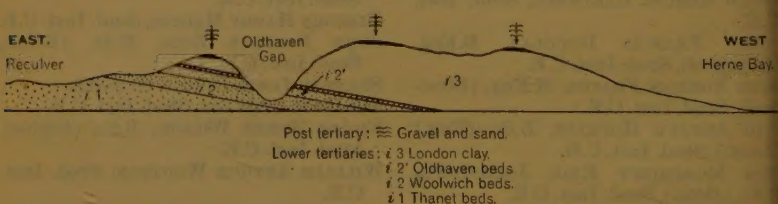
## Introduction.

The cliffs referred to in this Paper lie to the east of Herne Bay on the north coast of Kent. Their maximum height is approximately 100 feet above beach-level, and they slope towards the foreshore at slopes ranging from about 1 in $1\frac{1}{2}$ at the top, to about 1 in 4 at the bottom.

They consist of London Clay, capped in places with shallow deposits of sand and gravel. The clay is brown or blue or mottled in appearance and of a homogeneous texture, affording no indication of stratification with the exception of layers of septaria, or cement stones.

The total thickness of the clay deposit in the area is about 160 feet.

*Fig. 1.*




It overlies the Oldhaven beds (20 feet thick), the Woolwich beds (25 feet thick), and Thanet sand. The prevailing dip is in a westerly direction. A longitudinal section from Reculver to Herne Bay is shown in *Fig. 1.*

The cliffs have a characteristic tumbled appearance due to recurrent slipping. This slipping is not continuous from end to end, but takes the form of mud runs down the centres of valleys between relatively stable shoulders which project at intervals from the face of the cliff. This is clearly shown in *Fig. 2*, which is an aerial photograph of the cliffs taken about 5 years before the stabilization works referred to in this Paper were commenced.

During the summer and early autumn the cliffs remain comparatively hard and stable, but during the winter and early spring, particularly after heavy or protracted rainfall, masses of clay and overburden break away from the steep slopes at the top, disintegrating into pools of mud and slurry as they near the flatter slopes at the base of the cliff. The slurry surges over the retaining walls and breastwork on to the beach, whence it is washed away by the tide. At certain points where the disturbance is particularly deep-seated, as in the case of a persistent slide below Queen's avenue, all attempts to retain the toe of the cliff by means of a timber-piled breastwork have persistently failed.

In 1931 a scheme was carried out in connexion with the relief of unemployment, which took the form of constructing reinforced-concrete retaining walls embodying an under-cliff walk extending over a length of approximately 1,000 feet. Included in the design for the retaining walls were a number of reinforced-concrete bathing cabins, whilst substantial masonry pavilions and a café were also constructed on one of the lower terraces. The breastwork was strengthened by backing it with a continuous curtain wall of reinforced concrete, and the promenade was also surfaced with the same material.

In November 1935, a start was made with the construction of further lengths of retaining wall and bathing cabins but, after about 1 month's work, continuous and heavy rain was experienced and it was found necessary to suspend operations. The wet weather persisted and serious and extensive slipping began to take place. The upper retaining wall west of the shelter (A to B, Fig. 3, Plate 1) moved forward and partly overturned. The concrete decking between this retaining wall and the front row of bathing cabins, acting as a strut, transferred this movement to the roofs of the lower tier of cabins, which were partly overturned and badly damaged (Fig. 4, Plate 2). The promenade decking behind the sea wall burst upwards, and the disturbance extended to the foreshore, where an upheaval of blue clay occurred. The reinforced-concrete work which had just been commenced was damaged beyond repair, and an agreement was subsequently entered into between the Council and the Contractor for the cancellation of the contract.

Extensive movement also took place farther east in the vicinity of the other shelter (C to D, Fig. 3, Plate 1). The disturbance here, however, was not so deep-seated as in the area previously referred to and was confined to a layer of material not more than 10 feet deep, which spewed

over the top of the retaining wall and accumulated in a heap on the lawns and paths below (*Fig. 5*).

It was apparent that, unless prompt and effective measures were taken to check further movement, the stability of the whole of the lower promenade and adjacent structures would be endangered. In March 1936, the Council approached the Author's firm for an investigation and a report. One of the conditions laid down by the Council was that the report should deal with the area in sections which could be undertaken by the Council, independently of one another, but as part of a complete programme extending over a period of years.

## Preliminary Investigations.

From a preliminary inspection of the cliffs it was apparent that the saturation of the clay resulted not only from the infiltration and absorption of rainwater falling directly on the face of the cliff, but also, to a certain extent, from the percolation of underground water from the gravel and sand beds overlying the clay at various points along the top. The bulk of the cliff-face consisted of bare clay and mud, and whilst, in places, there was a sparse covering of weeds and coarse grass, this was insufficient to bind the surface together. During the summer months the cliff-face became riddled with cracks owing to the drying out and shrinkage of the surface clay. The cracking was further accentuated by surface movement resulting in the formation of fissures from 2 to 3 inches wide, extending down to depths of 6 feet or more below the surface, thereby enabling surface water to gain ready access to, and to saturate, the underlying clay.

Movement on an extensive scale usually accompanied the first heavy rainfall in late September and October. Masses of clay sheared away from the tops of the steeper slopes on clearly-defined planes, disintegrating into slurry as they neared the foot of the cliff. In the case of the worst slides, the degree of saturation of the material, by the time it had reached the lower retaining wall, was such that it was incapable of standing to a slope and continued to flow over the wall in a steady stream throughout the winter months.

Infiltration and absorption of water being the primary causes of the trouble, it followed that any effective method of intercepting some of this water, and leading it away harmlessly through defined channels, was bound to effect an improvement, if not a cure.

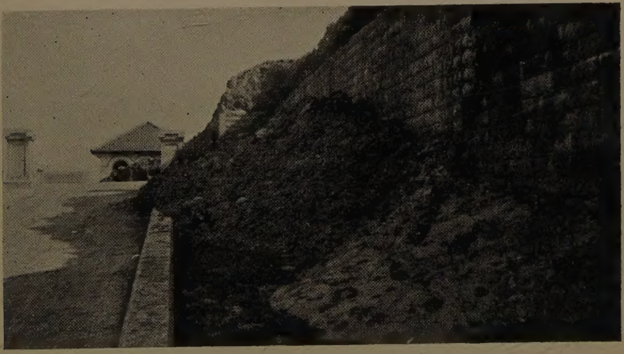
## Boreholes.

It was felt that it would be useless to attempt to formulate a scheme for remedial works and to arrive at an estimate of the cost, without first having obtained as accurate a survey as possible of the extent and depths of the water-bearing sand and gravel deposits at the top of the cliffs, as

*Fig. 2.*

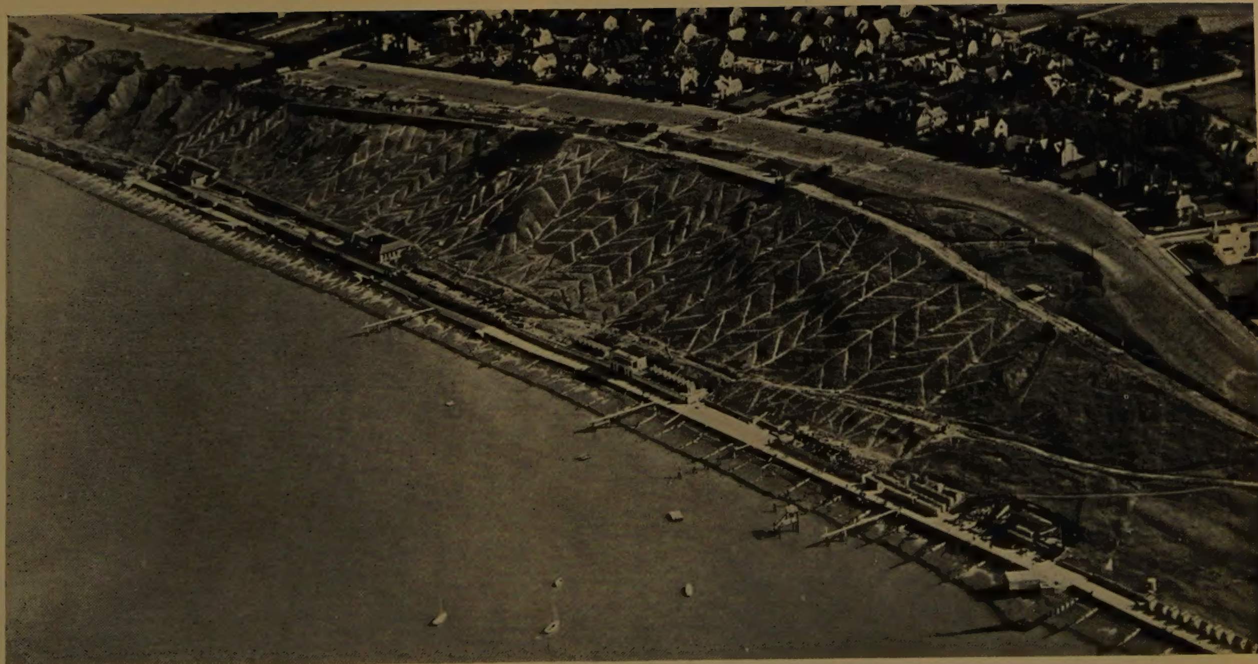
EAST CLIFF PRIOR TO THE COMMENCEMENT OF STABILIZATION WORKS.

*Fig. 5.*



CLAY AND MUD FORCED OVER THE LOWER RETAINING WALL.

Fig. 9.



EAST CLIFF, SHOWING THE WORKS NEARING COMPLETION.

well as an indication of the depths of unstable material on the axes of the principal landslides, and particulars of the underlying strata.

A contract was accordingly let for sinking fifty-two boreholes ranging from 10 to 80 feet deep. The positions of some of these boreholes are shown in Fig. 3, Plate 1. They were drilled in such an arrangement as to provide a continuous longitudinal record of the surface strata overlying the clay at the top of the cliff, and cross-sections of the cliff from top to bottom at various points.

Of a total of twenty-five boreholes sunk through the lawns at the top of the cliff, extending over a length of a little more than 1 mile, only eleven showed the presence of gravel and sand overlying the clay and, of these, only five showed seepage of water. This narrowed down the areas on which underground seepage towards the cliff-face was proved to exist to a length of 350 feet in the vicinity of Queen's avenue, and 500 feet in the vicinity of Hill Top road. The latter area was directly above the bathing cabins where the worst damage had occurred during the previous winter, whilst the Queen's avenue area had also been, in the past, the site of an extensive and deep-seated upheaval.

From the records of these boreholes it followed, therefore, that the direct seepage of water from the gravel-beds was responsible for only a small percentage of the slides. Where gravel and sand were not found the holes disclosed brown clay overlying blue clay, and the cores extracted from these holes showed this clay to be dense and unstratified.

Thirty holes were sunk through the face of the cliff and, although no underground water was tapped by any of them, except in the case of the deep holes at the foot of the cliff which were driven through to the underlying Oldhaven beds, swelling of the clay caused several of the holes to contract badly. There was every indication of a very moist condition of the clay down to considerable depths below the surface. Borehole No. 4, for instance, swelled at 18 feet below the surface, and borehole No. 12.W swelled between 22 and 30 feet down.

As was expected, apart from a layer of top deposit, consisting of material which had been carried downhill by landslides, the underlying material consisted wholly of clay down to the depths explored by the shallow 30-foot holes.

The depth of the slides varied considerably. On the line of boreholes Nos. 10 to 13, the depth near the top of the cliff was 3 feet, consisting of soil, mud, and brown clay with pebbles. Borehole No. 13, at the foot of the cliff on the same section, revealed soft dirty clay down to a depth of 9 feet, overlying dark brown clay which gave a hard core. The thicknesses of these slides, as revealed by the boreholes, were subsequently proved when the ground was opened up for the construction of the drains.

A section on the line of boreholes Nos. 3 to 7, where the worst damage occurred and where the trouble appeared to be most deep-seated, is shown in Fig. 4, Plate 2. The sandy bed below the lawns at the top of the cliff

was found to be 9 feet thick, and water was struck 9 feet down at the junction of the sand with the clay.

Borehole No. 7 at the foot of the cliff was drilled down to the Oldhaven beds, which were thus proved to be 70 feet below the surface. No seepage was encountered in sinking this hole until the Oldhaven beds were entered. At that point water was struck which subsequently rose in the borehole to within 27 feet 6 inches of ground-surface level (approximately half-tide level) where it remained stationary. An analysis of this water showed that it had no saline taste, and on evaporation left practically no residue of salt or chalk. The water was comparatively soft, being much softer than the town supply.

Further boreholes were sunk through the tops of the projecting shoulders, which form so conspicuous a feature of the cliffs in this area, in order to determine whether there existed any physical difference in the characteristics of the material of which they were composed which would account for their marked stability compared with the remainder of the cliffs.

One of the holes showed brown shaly clay with pockets of sand and claystones to a depth of 7 feet, and, underneath this, dark brown clay (slightly moist) for a further 15 feet to the bottom of the borehole. Another hole showed 12 inches of topsoil overlying mottled clay and brown clay down to the depth explored, namely 28 feet 6 inches. The clay was moist down to 9 feet. From 15 feet downwards the hole showed a tendency to swell which increased with the depths, but the clay was particularly tough and the hole was sunk without water, except for the purpose of taking cores.

Reference has been made to the Queen's avenue area, where conditions during recent years have been particularly bad. A section through boreholes Nos. 24.S to 20 in this area shows that water was struck in the gravel beds at the top of the cliff at a depth of 5 feet below ground-level.

Near the foot of the cliff the records from borehole No. 21.S indicate that the ground has been subject to a major disturbance down to a depth of 40 feet below the surface. Water was struck 38 feet down in a bed of dirty brown sand. The borehole entered the Oldhaven beds 65 feet down at which level water was again struck which rose and remained stationary at a depth of 38 feet below ground-level; this was exactly the same level as in the case of the deep borehole No. 7 about 2,000 feet farther west. Between borehole No. 21.S and the breastwork a pronounced ridge exists which projects to a height of nearly 15 feet above the surrounding surface and appears to be the result of a great upheaval, the trough left by the upheaval having subsequently been filled by detritus brought down by further landslides from above.

## Conclusions from the Borehole Tests.

The conclusions arrived at from the borings, and the proposals embodied in the Engineers' report to the Council in October 1936, were as follows :—

(1) *Boreholes along the top of the cliffs.*

As previously stated, the presence of sand and gravel beds containing water was discovered to be confined to two areas, shown hatched in Fig. 3, Plate 1. Percolation of water from these deposits through the face of the cliff contributed towards the saturation of the ground, and it was realized that an improvement would be effected in these areas if steps were taken to intercept this underground water before it reached the cliff-face.

The recommendation therefore included a proposal to construct a pipe and rubble drain along the top of the cliff at a safe distance landward from the edge and to depths which would be determined by reference to the boreholes. The drain was to be laid to fall towards three junction-chambers, from which water would be conveyed down the face of the cliffs by lines of pipes.

It was realized that there were possibly other underground water-courses draining towards the cliff whose existence had not been detected by the boreholes. For this reason borehole No. 32 was sunk immediately behind an observed seepage through the cliff face, but no water was tapped. A continuous drain would act as a barrier, intercepting all these underground watercourses, and leading the water away to the outfalls.

(2) *Boreholes in the face of the cliffs.*

The records of the boreholes sunk through the face of the cliffs, several of which were located on the axes of the more serious landslides, indicated that, in general, the thickness of the slides increased from a depth of 2 or 3 feet at the cliff top to 10 to 15 feet at the foot, and that the underlying material consisted of a fairly homogeneous undisturbed clay with a varying moisture-content. In the case of the Queen's avenue area the possibility of moisture having been absorbed by capillary attraction from below was not overlooked, but in the case of the cliffs behind the retaining walls, owing to the dip, the underlying water-bearing sands were situated at such depths that it was considered unlikely that the stability of the overlying clay would be adversely affected by them.

The boreholes sunk through the projecting shoulders showed that they did not differ internally from the structure of the London Clay elsewhere in the area. Denudation of clay cliffs at other parts of the English coast has been found to proceed on similar lines, as for instance, at the Warren, Folkestone. Apart from a dry outer crust of hard shaly clay, the

shoulders consist of dense brown clay containing an average quantity o moisture.

The effect of these shoulders was to divide the cliff into a series of natura drainage-areas, each of which contained one or more separate anc distinct slides, increasing in depth from top to bottom. The depths of th slides also varied laterally from zero, at the sides of the shoulders, to a maximum on the axes of the slides. In many cases, and particularly afte heavy rain, clearly-defined planes developed along the sides of the shoulder along which sliding occurred. These planes of sliding, together with ex tensive transverse cracks which formed in the surface of the slides, partl as a result of the continuous movement and partly as a result of shrinkag when drying out, provided the means whereby water gained access to th underlying clay, thereby helping to destroy its cohesion.

To achieve a solution of the problem, it appeared to be necessary :—

(1) To construct a series of deep primary drains from top to botton of the cliff, located centrally along the axes of the worst slides and carrie down into stable material below the base of the moving mass.

(2) To construct "herring-bone" laterals at close spacing, feeding th primary drains and bottomed up in stable material, which, apart fron draining the intervening spaces, would also have the effect of dividin the cliff-face into small areas and breaking up the continuity of th slides.

(3) To construct interceptor drains, located along the lines of the plane of sliding previously referred to, which would serve the purpose of inte cepting, at surface-level, water draining off the steep slopes of the pro jecting shoulders after heavy rain.

(4) To construct a deep longitudinal drain at the foot of the cliff behin the retaining walls to receive the discharge from the primaries and conve it to outfall chambers, and thence, by means of outlet pipes under the retai ing walls, to the foreshore.

(5) To dress down the projecting shoulders and all excessively stee slopes to flatter gradients capable of being permanently stabilized b means of drains.

(6) To fill all large cracks with properly consolidated material, to lev out inequalities in the surface, to fill hollows or depressions in which wat might tend to accumulate, and to grade the surface of the ground towar the drains.

(7) To spread top soil over the entire area, to fork it over, and to seed with selected grasses and weeds.

(8) To reconstruct the damaged upper promenade along the top of t cliff in the form of a piled timber staging.

After due consideration, the Council decided to apply for loan sancti to the expenditure of a sum of £31,000 to cover the cost of the upp

longitudinal drain and the drainage of the whole of the cliff-face west of Sea View road, that is to say, a length of 1,700 feet. In June 1937, the engineers were instructed to proceed with the preparation of plans and specifications so that tenders could be called for by the beginning of July.

## The Works.

The upper longitudinal drain (Fig. 3, Plate 1) was constructed at a minimum distance of 40 feet from the edge of the cliff, the total length from the east to west outfalls being 3,900 feet. The excavation was of a uniform width of 3 feet, and varied in depth from 5 to 12 feet. The drain was designed with a minimum gradient of 1 in 120, the invert being located at such a depth as to be at no point less than 18 inches below the top of the solid clay underlying the sand and gravel deposits. It is divided into lengths of approximately 270 feet by inspection-pits, 4 feet 6 inches in internal diameter, fitted with ladders and covers, each pit containing a sump 12 inches deep to serve as a silt trap.

The drain itself consists of a single row of 12-inch diameter porous concrete pipes with non-porous inverts which were laid with open joints in a narrow channel excavated along the bottom of the trench; they were then covered with brick hardcore to within 12 inches of the surface. At this level the hardcore was consolidated by tamping to a compact surface and was covered with a 3-inch filter-layer of clinker. Soil was then back-filled approximately to surface-level and the turf was replaced and rolled.

At the commencement of the work a layer of clinker was deposited around the pipes to a depth of 6 inches above the top, but this was discontinued owing to the difficulty experienced in obtaining consignments of dust-free clinker.

The area of the watershed which drains towards the cliffs is comparatively small. At its widest point, opposite Sea View road, it does not extend more than 1,800 feet back from the cliff.

Beyond this the land falls away towards a small stream which drains eastward in the direction of Reculver. The total area of watershed lying to landward of the upper longitudinal drain, between its highest point at Burlington drive and its outfall at Sea View road, is only 50 acres. A comparatively small portion of this land is built upon, but over the greater part rain is free to percolate into the sub-soil. A drainage coefficient of 1 inch in 24 hours applied to the area under consideration, making an allowance for roads and buildings drained directly to the Council's sewers, gives a run-off of approximately 2 cusecs, which is well within the capacity of the drain.

Considerable difficulty was experienced in keeping the porous pipes clean while the work was being carried out during the winter months. Unless they were carefully watched, workmen with muddy boots would walk

along the tops of the pipes, forcing clay into the pores. The trouble was eventually overcome by covering the pipes with straw matting as soon as they were laid, and by placing planks along the top to serve as gangways.

The excavation of the trenches throughout the greater portion of the drain was carried out by mechanical excavator. Where the trench was driven through clay the sides stood up satisfactorily, and timbering was subsequently inserted without difficulty. Considerable trouble, however, was experienced when driving the trench through the sand and gravel beds, as the sides would cave in before timbering could be fixed; here it was found necessary to continue the work by hand.

After completing a length of drain between inspection-pits, a narrow channel was excavated by hand along the bottom to receive the pipes, the material taken from the channel being tamped against the lower portion of the pipes and finished with a fall towards the pipes from both sides of the trench.

Three outfall pipe-lines were provided from the upper longitudinal drain to the foreshore. The central and western outfalls were laid on a part of the cliffs which was subsequently stabilized by draining and gave no trouble.

The cliff below the easternmost outfall, however, lay outside the area which was to be treated under the contract, and it was realized that extensive movements in the cliff-face were to be expected on the line of this outfall. In order to allow the pipes to adjust themselves to the changing contours of the cliff, 6-inch-diameter steel tubes were used with "Victaulic" joints. The pipes were laid on a bed of hardcore carried down to a minimum depth of 2 feet below ground-surface level. The pipe-line was secured at its upper end to a substantial concrete anchorage and the lower end of the line was threaded loosely through a hole in a concrete anchor-block and secured by hardwood wedges driven in from the back. Setting-pieces were inserted at changes of gradient to give greater flexibility.

During the winter following the completion of the works further extensive landslides occurred on the site of this outfall, and the upper lengths of pipe became buried under several tons of slurry. The extent of the movement may be gauged from the fact that the lower portion of the line was drawn out of the bottom anchorage and dragged uphill for a total distance of 12 feet. In spite of this, however, the outfall continues to function efficiently.

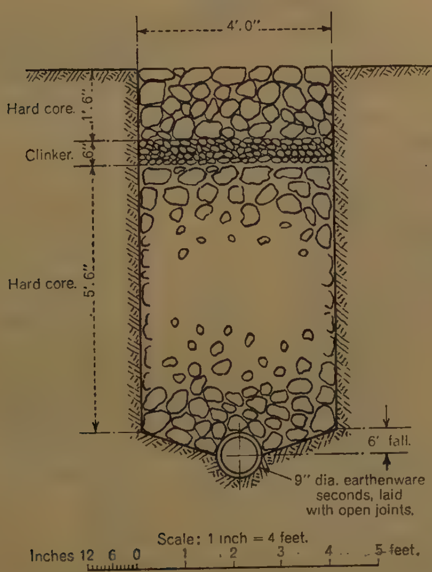
## Drains in the Face of the Cliff.

The subdivision of the cliff drainage system into primary, lateral, and interceptor drains, together with a lower longitudinal drain along the back of the retaining walls, has already been referred to. A typical layout of a section of the work as executed near the eastern end of the site is shown i

Fig. 3, Plate 1. The primary drains, generally, were constructed to even gradients between inspection-pits which divided each drain into three or four lengths.

The drains were 5 feet deep at the top of the cliff and it was rarely found necessary to exceed a depth of 10 feet at the bottom. They were constructed to a uniform width of 4 feet, and a single row of 9-inch diameter earthenware seconds pipes was laid, with open joints, along a narrow channel excavated along the invert, the bottom of the trench being dished to provide a proper fall from the sides towards the centre. The trenches were filled with brick rubble to within 2 feet of the ground surface, at which level the hardcore was tamped and covered with a clinker seal 6 inches

*Fig. 6.*




thick (see *Fig. 6*). The drain was then filled with a further layer of rubble to surface-level, the surface being knocked over with a hammer and tamped.

The "herring-bone" lateral drains were constructed 3 feet wide and not less than 4 feet deep, and were filled with rubble to ground-surface level. The average distance apart of these drains ranged from 15 to 25 feet, depending upon the slope of the ground. The interceptor drains excavated along the edges of the slides were made 3 feet wide and 3 feet deep.

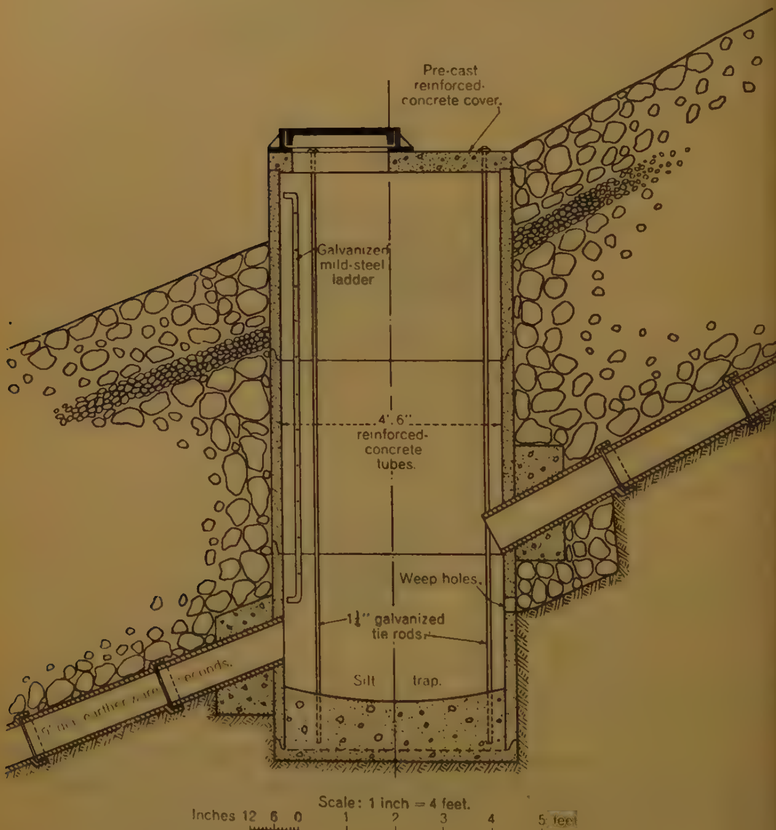
At the lower ends of the primaries, where solid clay was, in places, overlain by a considerable thickness of soft unstable material, the laterals were made deeper, so as to bottom up in stable material. Wherever the

depth of lateral exceeded 5 feet, 6-inch pipes with open joints were laid along the inverts.

The lower longitudinal drain, constructed a few feet back from, and parallel with, the reinforced-concrete retaining wall, was made 4 feet wide and varied in depth from 10 to 14 feet, depending upon the depth of the drift and the varying ground-surface level.

12-inch diameter porous pipes with concrete inverts were laid along

*Fig. 7.*




the trench at a minimum gradient of 1 in 120, and inspection-pits were constructed at its intersection with each primary drain. The inspection-pits were built up of 4-foot 6-inch (internal diameter) reinforced-concrete tubes with ogee joints, fitted with pre-cast reinforced-concrete covers and secured to the foundation concrete by means of long 1¼-inch diameter galvanized tie-rods (*Fig. 7*). The purpose of the tie-rods was to allow the inspection-pits to take up a certain amount of tilt without fracturing

ı the event of further movement occurring in the face of the cliff after rection.

Holes for the drain-pipes were broken through the walls of the pits, and veep-holes were also drilled through the walls, under the drain inlets, to llow for the filtration of any water percolating along the outside of the ipes. Concrete collars were constructed around the drain-pipes where hey passed through the walls of the inspection-pits, in order to prevent any endency on the part of the pipe-lines to move downhill.

Some difficulty was at first experienced in reconciling the Council's lesire to obviate any inconvenience to the visiting public at the height f the season with the Contractors' desire to take full advantage of avourable weather conditions for prosecuting the work with the utmost rigour.

Fortunately the Council took a sympathetic and broad-minded view of he Contractors' difficulties and allowed the work to proceed without nterference throughout the season.

The excavation of the cliff drainage-system was carried out by hand, he material being thrown into shoots which were kept lubricated by means f a steady stream of water. At the bottom of the cliff the material was lischarged into Decauville trucks and run to dump. For a time, during the vinter months, much of the excavated material was dumped into the sea, but it was found that in calm weather it tended to solidify, and for this nd other reasons such a method of disposal was stopped.

The principal disadvantage of shoots, as a means of conveying excavated material from the trenches to the trucks at the foot of the cliff, vas the dirt and slurry which always accumulated on the roof of the bathing abins when excavation was in progress. In addition, there was a tendency or lumps of material to fly off the shoots at sudden changes in gradient, nd clog drains which had been previously filled with rubble. This trouble vas eventually overcome by covering drains under the shoots with hessian r tarpaulins. On the other hand, the flexibility of a system which allows xcavated material to be disposed of from almost any portion of the cliff, vithout double handling, has much to commend it from a contractor's oint of view.

The top soil from the drains was not dumped on to the shoots, but was ipped in heaps alongside the drains and subsequently spread over the round to provide a suitable surface for seeding.

Excavation in the face of the cliffs was commenced at the eastern end f the site with the object of safeguarding the retaining wall against further lamage. The mess and discomfort endured by the men during these early tages of the work can well be imagined, whilst after 3 months of winter vork the excavation of deep trenches became fraught with a certain amount f danger. Work at the eastern end was then discontinued and the Contractors transferred their activities to the western end, where the cliffs were nore stable.

Banks of clay were built up against the outer face of the retaining wal with material shovelled over from the back, as a measure of protectio while work was in progress elsewhere. This had the effect of counter-balancing the outward thrust, and allowed successive slides to flow harm-lessly over the top of the wall. On the completion of the drainage work these banks were dug out; the wall was then cleaned up, and was foun to have sustained no further damage.

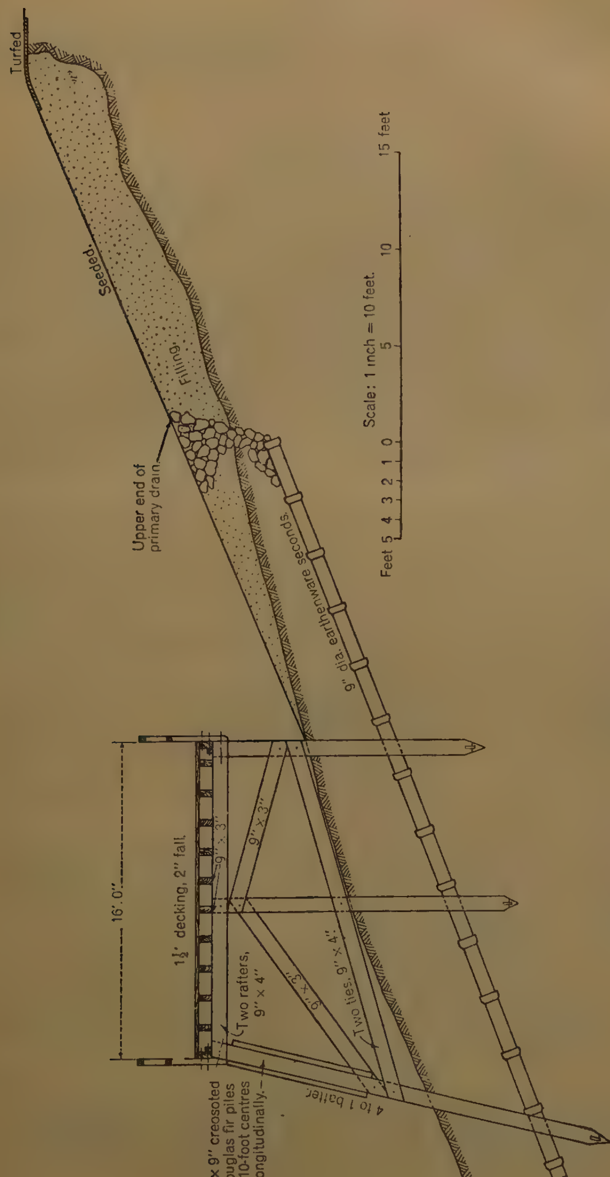
All excessively steep slopes, as well as the steep-sided shoulders whic projected at intervals from the face of the cliff, were dressed down by han to flat, even slopes, and the excavated material was either sent to dump o used for filling in hollows and depressions. An attempt was made to wor to a maximum gradient of 1 in 3. In one or two cases, however, it wa found that this would have entailed excessive excavation, and the slope were dressed down to gradients of 1 in 2; no subsequent movement wa detected.

At the eastern end of the works the promenade which formerly existe along the top of the cliff had been totally destroyed by successive landslides It was thought to be inadvisable to attempt to reconstruct this promenad in its original solid form, as this would have involved depositing a heav weight of filling near the top of the steepest and most unstable portion o the cliff.

The cliff steps, leading from Sea View road to the sea-wall and lowe promenade, afforded a convincing illustration of the stabilizing effect of piled structure upon the underlying soil. Although the cliff-face on bot sides of the steps was constantly on the move, the steps sustained ver little damage, and the underlying soil remained comparatively stabl throughout. It was, therefore, decided to reconstruct the promenade i the form of a braced timber staging with an outer row of raking pile strutted back to one or two rows of vertical piles, depending upon th configuration of the cliff (*Fig. 8*). The primary and lateral drains wer extended underneath the staging and were carried well into the upper edg of the cliff, which was dressed to an even slope and turfed over.

After spreading top soil from the drain excavation over the surface o the cliff the entire area was raked over and seeded with selected grasse mixed with an equal quantity of coarse grasses and clover, at the rate o 5 cwt. per acre. An analysis of the selected grass seed showed that i contained tall oat grass, cocksfoot, tall fescue, perennial rye grass, sea lym grass, marram grass, various clovers, etc. The coarse grasses consisted o clovers, weeds, etc., which were mainly cleanings from various seeds. Th seed germinated well, in spite of the rather poor quality of the soil, an a few weeks after sowing the entire slope was evenly carpeted in gree presenting a striking appearance in contrast with the uneven brow muddy appearance of the untreated portion farther to the east.

*Fig 8.*




SECTION THROUGH THE UPPER PROMENADE STAGING AFTER CONSTRUCTION.

## Costs.

The work was completed in October 1938, at a total cost of approximately £31,600, which works out at approximately £17 10*s*. per linear foot of cliff.

## Conclusion.

The cliffs were kept under careful observation during the winter months, during which time rainfall was above the average, and, with the exception of two small local slides which occurred after a series of very wet days in October 1939, and which were stabilized at an expenditure of about £350, no trace of movement or slipping of any kind has been detected.

*Fig. 9* (facing p. 415) is an aerial photograph taken when the works were nearing completion, after the seed had taken root.

The discharge from the outfalls is continuous, but varies between wide limits. Measured discharges from the outfall from inspection-pit No. 7 at the foot of the cliff, which deals with the drainage from 3 acres of cliff, are shown in Table I.

Table I.

| Date. | Conditions. | Discharge: gallons per hour. |
|---|---|---|
| 25th November . . | 3 hours after heavy rain lasting 3 hours. | 600 |
| 16th December . . | After a night drizzle. | 110 |
| 3rd January . . . | After a morning drizzle. | 450 |
| 12th January . . . | During frosty weather. | 90 |
| 14th January . . . | During frosty weather. | 40 |

From September 1937 to March 1938, the Engineers were represented at the site by Mr. F. Lorimer Bruce, Assoc. M. Inst. C.E., and from March 1938 until the termination of the work, by Mr. G. E. B. Coulcher, M. Inst. C.E., who acted as Resident Engineer.

The Contractors for the drainage Contract were Messrs. J. W. Ellingham Ltd., of Herne Bay, and the Contractors for the boreholes were Messrs. Legrand, Sutcliffe & Gell, Ltd., of Southall. The seeds were supplied by Messrs. Sutton and Sons, Ltd., of Reading.

The Paper is accompanied by five sheets of drawings and three photographs, from which Plates 1 and 2, the Figures in the text, and the half-tone page-plates have been prepared.

## Discussion.

**The Author,** in introducing his Paper, showed a number of lantern-lides, illustrating the conditions at Herne Bay prior to the works described, he damage caused by the land-slips, and the progress of the works.

He observed that since the work had been completed it had been rranged that weekly inspections should be carried out during the winter nonths, and that any cracks which developed should be filled with clay, horoughly tamped, and turfed over, since even a small crack on one of he steep slopes might, if neglected, develop into something much more erious.

The omission from the Paper of any specific reference to the science f soil-mechanics was not due to any lack of appreciation of its value. Collaboration between the research worker and the practising engineer vas receiving the attention of the Earth Pressures Committee of the nstitution Research Committee. It was to the advantage of the engineer o be able to draw upon the accumulated and specialized experience f the research worker, whilst it was equally to the advantage of the esearch worker to have access to works and to become acquainted with the ractical difficulties confronting the engineer, having regard to methods f execution, site conditions, and, perhaps most important of all, limitations f financial resources.

**The President** said that it was very useful, in the case of a new sub-ect like soil-mechanics, to have a practical demonstration such as that iven in the Paper, which enabled scientists and engineers to learn from ne another.

The great change which had taken place on many parts of the coast ad been due, he thought, very largely to the imposition of artificial onditions on land that had once looked after itself. The drainage, which formerly did its work for an agricultural region, was completely pset, and the surroundings were often in serious danger of being ruined. Ie thought it was quite clear that, but for the help of the engineer, the menities of Herne Bay would have been completely spoilt.

**Mr. F. E. Wentworth-Sheilds** believed that the Building Research tation was developing a technique by which it would be possible to take uickly samples of any desired strata, and to test them in such a manner hat they could be classified and related to other earth samples; indeed, hey could be given labels or figures to indicate the strength of the clay r other material, its power of absorbing water, and so on, and it would

be possible to tell fairly quickly and readily whether one clay would behave like another. Thus something much more useful would be stated than the fact that the clay in question was dark brown or brown mottled and its probable strength and behaviour would be known. Perhaps the Author might be able to furnish some information of that kind.

On p. 421 it was stated that the "herring-bone" lateral drains which led into the main drains were not less than 4 feet deep. Why had the Author considered such a depth to be necessary? He appreciated that the primary drains should be of sufficient depth to get right down to the bottom of the disturbed clay and on to the stable bed, but the real function of the lateral drains was to cause the rainwater which fell on the slopes to pass quickly into the primary drains; for that purpose a surface drain would have sufficed. The most important thing was to prevent rain from soaking into the clay and so softening it, and the Author had pointed out that cracks had to be prevented from forming, and that any which formed had to be filled. Therefore it would appear that the principal aims of the engineer should be to cover the clay with grass and to provide sufficient surface drains to ensure that the ground did not become softened by water percolating into it.

**Mr. George Ellson** observed that the work appeared to have withstood successfully the most trying winter within living memory, so far as the effect of weather upon earthwork was concerned. The cliff-face at Herne Bay, apart from the area wherein deep-seated trouble was apparent, appeared to suffer more from scour and weather erosion than from the more serious slides which were usually encountered on cliff-faces and in clay cuttings, and it might be that the trouble had been accentuated by the prevalence of easterly and north-easterly winds during the winter of 1935 and 1936 and subsequently.

The works which had been carried out might fairly be described as surface-drainage works, and the results obtained so far appeared to indicate that the Author's diagnosis of the trouble was correct over the main area. Time alone would show whether the remedies adopted would be ultimately successful in the length where the bulge in the lower promenade and the uplifting of the sea-bed had taken place.

If the core at borehole No. 7 had been carefully taken out, in accordance with the latest methods advocated by Professor Karl von Terzaghi, M. Inst. C.E., and the Building Research Station, the plane of the slip at that section might have been revealed, and it would then have been possible to form a rough idea of the depth of the disturbance in that area. Indications as to whether or not the clay was fissured might also have been obtained; information on that point was of considerable value in studying remedial measures. The Paper did not indicate whether or not there were clearly-defined slip surfaces on other areas of the cliff-face and he concluded, therefore, that the principal trouble was weather erosion of the surface, owing to the percolation of water through cracks

ormed during the summer, resulting in the conversion of the clay down o a certain depth into slurry, and the subsequent flow during the winter. The lantern-slides exhibited by the Author rather confirmed that view, and he would like to suggest that observations should be made on reference-points fixed in the solid below the slurry and their positions hecked. He considered also that it would be wise to cut a deep trench hrough the middle of the bulge and to examine it closely for signs of the presence of a deep-seated slip. That would be easy in summer, but would be very difficult and costly in winter after a deep-seated slip had occurred.

Mr. Ellson exhibited a number of lantern-slides referring to slips of which he had had experience. The first of them took place in the Wealden Clay at Sevenoaks on the Southern Railway on 13th April, 1939, on the up side of the line just below Sevenoaks tunnel. The slip was about 180 feet long, in a cutting approximately 60 feet deep at that point. The railway was constructed about 70 years ago, and the sides of the cutting had an average slope of about 1 in 2. Trouble had been experienced about 21 years ago in the same cutting, and had then been overcome by cutting vertical trenches through the slip to a small depth below the slip surface, those trenches being paved with concrete and filled with hardcore.

When the 1939 slip occurred, he took the opportunity of consulting Professor von Terzaghi, who expressed the view that, with a water-content close to the liquid limit, slides in fairly homogeneous soft clays were exclusively due to the low shearing resistance of the material in its original state. Such slides always occurred during, or shortly after, a cut was excavated. Slides in stiff clays never occurred immediately after a cut was made, and the time which elapsed between the excavation of cuts and the formation of slides was very variable. Slides in stiff clays were preceded by a gradual disintegration of the slope material, owing to a gradual swelling of the clay which proceeded from the percolation of water into cracks and other fissures. The final product consisted of fragments of hard clay embedded in a soft matrix, and with the increasing weight of the overburden the ultimate water-content of the soft matrix decreased rapidly; consequently, in its final state the clay was far from homogeneous. That explained the fact that the surface of sliding for stiff clays very seldom coincided with the surface of sliding computed for homogeneous materials. Immediately after the Sevenoaks cutting had been made the factor of safety of the slopes against sliding might have been as high as 5 or 10, depending upon the depth of the cutting. At the point where the slip occurred that depth would be about 60 feet, whilst it decreased to zero about $\frac{1}{4}$ mile farther south. A slip corresponded with a factor of safety of 1, and the characteristics of the slides which took place in stiff clays determined the remedial measures to be adopted in dealing with such slips. It might not be economical to make the slopes of cuttings flat enough at the start to ensure stability for the ultimate stage of a process

which might extend over many decades (actually in the case in question it took 70 years), and that was the position which had to be faced with regard to many existing railway cuttings.

Mr. Ellson had decided that at Sevenoaks the procedure which had been adopted 21 years previously, and which had proved generally effective, should be followed in a modified manner. Four hardcore drains were constructed in the standard manner adopted on the Southern Railway, trenches being first cut up the slope to below the plane of the slide. As those trenches were cut a very clear indication of the slide-surface was obtained, and no difficulty was experienced in deciding when the bottom of the slide was reached. The bottom of the trench was paved with concrete laid to proper falls, and the trench was filled with hand-packed brick rubble. The trenches were regarded as valuable buttresses to the sides of the clay cutting, as well as acting as a drain. They cut the slip material into sections, so that the slip could not move again as a whole, and they buttressed the back of the bank against further movement. It had been found by experience that a surfacing 6 inches in thickness, consisting of ash mixed with a small proportion of clay, would minimize the surface cracking in hot weather.

Professor von Terzaghi had expressed the view that the berm which was present about the middle of the finished slope tended by its nature to stabilize the slope, and he had furnished a very full explanation of his reasons for holding that view. It should be realized, however, that once a slip had taken place the surface of the slope had been weakened for all time; hence the necessity for taking effective remedial measures. Although the slip took place during a period of dry weather—and Mr. Ellson had never before seen a slip take place when the surface was in such good condition—the decisive factor was the progressive swelling and softening of the clay. In addition to constructing the hardcore drains, he had also trimmed back the tops of the slopes of other parts of the cutting to a gradient of 1 in 3 for a distance of about 25 feet from the top of the slope, because he thought that would lighten the pressure on the lower clay.

A large fall of chalk took place in Folkestone Warren early in 1940; the cliff-face at the point concerned was about 560 feet above sea-level, and the slip covered a very large area. On that occasion about 100,000 tons of chalk slipped down and spread out, filling a hollow left by some previous slip. The fall was typical of other falls that had occurred not only last winter, but also in other years on that coast; but the very severe weather conditions of last winter had caused considerably more falls than usual. A rainfall of about 23 inches occurred over the area in question during the last 3 months of 1939, and the chalk became super-saturated with water. The severe frosts caused disintegration of the surface, as at Herne Bay, and also doubtless influenced the expansion of water located in fissures at the back of the face, such as were prevalent along the tops of

chalk cliffs. He thought that the probable cause of the preliminary fractures was the expansion, in the unrestricted seaward direction, of the earth's surface in hot weather, and the drying effect of the sun in summer. Possibly material fell down into the cracks, and when the cold weather came contraction did not take place; the part which had moved then stayed where it was, and gradually it became a little unstable. He thought that that was the cause of the chalk-cliff falls experienced on the south coast. Moreover, chalk cliffs included natural beds along which falls could take place more easily; for example, the cliff-face at the back of the fall in question, from which the slide had taken place, had a smooth appearance. After the fall had occurred some small subsidences in the top were watched very carefully for a considerable time, but the face appeared to have become stabilized again.

Mr. Ellson concluded by showing lantern-slides illustrating slips at Winchfield and Botley, which had been referred to by him on a previous occasion[1].

**Sir Cyril Kirkpatrick**, Past-President, thought that the Author had been fortunate in being able to take sufficient borings to ascertain the seat of the trouble. His own experience was that many councils and people who had the power to spend money strongly objected to borings being taken, so that it was very difficult to obtain a good survey before commencing operations. By means of the twenty-five borings at the top the Author had been able to locate the seepage of water over a comparatively small area; that would have been impracticable if only a few borings had been taken. He would like further information from the Author with regard to the type and construction of the 12-inch diameter porous concrete pipes with non-porous inverts mentioned on p. 422. He would also be glad if the Author would indicate the positions of the rain-gauges from which the information regarding the rainfall, given in Table I, was obtained. The run-off seemed to be small, and it would be of interest to have some idea of its quantity in relation to the rainfall.

**Mr. Raymond Carpmael** considered that two important points brought out in the Paper were, firstly the futility of relying on toe-walls *per se* to afford sufficient support to clay cliffs, and secondly the primary necessity of providing an efficient drainage-system. He would like to congratulate the Author and his firm upon a well-thought-out scheme, upon the successful manner in which the work had withstood an abnormal winter, and upon the close resemblance between the estimates and the cost of the works. In works of such a character certain unknown quantities existed, and a considerable percentage allowance was usually made for contingencies; it would be interesting to know what provision had been made at Herne Bay.

---

[1] *Discussion on* "Engineering Problems Associated with Clay, with Special Reference to Clay Slips." Journal Inst. C.E., vol. 8 (1937–38), pp. 481–482 (April 1938).

It was a wise precaution to drive boreholes. In that respect the Autho was much more fortunate than railway engineers, who often had to do th work first and subsequently try to discover the cause of the disturbance The boreholes had revealed, not so much what should be done, as what wa unnecessary, and in that respect they had prevented needless expenditur of money and labour. As he understood it, the work in question wa practically all shallow work on sliding surfaces. His experience wa that the cause of movement was nearly always the sliding of super incumbent material on a soapy film which had been softened by th impregnation of surface water; that film overlay more stable material and was probably composed of disintegrated material which had passe through cracks during dry weather until it was held up by more soli strata. The trenches of primary drains should be founded below the leve of that film or plane of cleavage, which some railway engineers rathe loosely described as "the slip."

He had experienced considerable trouble in the past with clay slope in the Severn valley between Shrewsbury, Bridgnorth, and Bewdley where continuous slips had occurred over some sections. By arrange ment with the landowners, men had been allowed to inspect the slope periodically, sometimes going as much as ½ mile up the hill and tramp ling down very carefully all cracks, with the object of preventing infiltra tion of water into the slopes below. He considered that the importanc of that precautionary method of closing all cracks could not be over emphasized.

In his practice he did not normally find it necessary to lay pipes a the bottom of what were described as main drains, which were usuall formed with heavy slag to provide support as buttresses as well to ac as drains. Shallow inclined "herring-bone" lateral drains were doubtle useful as shallow surface drains, but they could not possibly afford an support in case of further movement.

**Mr. L. F. Cooling** observed that it might be of interest to outli present knowledge of the mechanism of slides in the type of soil und consideration. London Clay belonged to a group of clays which, as th result of having been subjected to a very high overburden pressure in th course of their geological history, now existed at a low water-conten approximately equal to what was normally termed in soil-mechanics th "plastic limit." That type of material was defined by Professor vo Terzaghi[1] as a stiff, fissured clay. Such clays were very dense and stro in the body of the material, but they contained a network of fissures whi formed a potential source of weakness. London Clay, for instance, ha probably been at one time under an overburden pressure of the order of tons per square foot, and, although erosion had removed most of the ove

[1] "Stability of Slopes of Natural Clay." Proceedings, International Confere on Foundation Engineering, vol. I. Harvard, 1936.

rden, the clay now existed in a dense condition. The moisture-content samples, expressed on the basis of the weight after drying at 105° C., nged from 25 per cent. in a hard clay to probably 33 per cent. in the rface layers, where there had been an opportunity of expanding; at represented a porosity, measured on the total volume, of between and 48 per cent. The body of the material was exceptionally strong, e compressive strength of unrestrained cylinders ranging from about lb. per square inch up to as much as 100 lb. per square inch. The sures, however, exerted a very important influence upon the behaviour London Clay in cuttings, and rendered it very prone to troublesome des, even on comparatively flat slopes. When the clay in its natural ate was situated at a depth below a level surface, the fissures remained osed, owing to the high lateral restraint, and had little influence upon e soil properties; but when the clay was located near the slope of a tting, the removal of the lateral support caused uneven expansion, which d the effect of opening the fissures, thus weakening the structure. Moreer, because the body of the material was so strong, the fissures could main open for a depth corresponding with the expression :—(compresve strength of material)/(weight per cubic foot); thus, in all probability, ey could remain open to a depth of 24 feet and more. As a consequence the opening of the fissures, the overburden was carried on a smaller aring-area; the shearing resistance was thus reduced, and with it, of urse, the stability of the slope. During rainstorms water accumulated the fissures, and the clay could then swell along the walls of the fissures nder zero pressure, even when they were located at an appreciable depth low the surface. That process of non-uniform swelling weakened the aterial still further by forming cracks. In addition, drying shrinkage curring at the surface produced cracks, and when those cracks became led with water during rainstorms, the hydrostatic pressure in them inoduced an additional disturbing force, so that the stability of the slope as less than ever after a rainstorm. Once a slide had begun, the lubrited walls of the fissures came into contact and the water was trapped in e fissures. The resistance was then reduced considerably, so that once slide had started it was liable to proceed quickly. In addition, the rface soil was affected by the normal weathering processes, such as frost, hich broke up the surface.

Slides in such material as London Clay represented failures along zones local weakening within a very deep zone of potential disintegration. was interesting to observe, from the Author's account of the failures served in the Herne Bay cliffs, that, although the majority of them opeared to be shallow slides, deep slides had occurred at various points, pparently coinciding with portions of the cliff which included a wateraring stratum at the top. With such a mechanism of failure, surface ainage and measures to reduce shrinkage-cracking at the surface of the il would obviously serve to increase the stability of the slope, particularly

against shallow slides and during periods of adverse weather condition when the factor of safety might be reduced below unity.

The Author had mentioned the advantages, in considering practic problems, of supplementing site observations by quantitative measur ments of soil properties. Mr. Cooling appreciated the fact that the bor hole samples mentioned in the Paper were examined prior to Octob 1936, which was early in the history of soil-mechanics in Great Britain but much more valuable information could have been obtained from th samples. The only descriptions of the soil contained in the Paper were a purely qualitative nature. Although London Clay was fairly wel known, and a description conveyed a general impression to those famili with it, that was not sufficient for identification purposes; substanti variations might be revealed between samples from different parts of th formation. Simple quantitative tests were now available for the purpo of identifying a soil, and if the borehole samples had been tested to dete mine the natural moisture-content, and the liquid and plastic limits, ar had been subjected to unrestrained compression tests, it would have bee possible to record a much more definite idea of the type and condition of th soil and of the degree of its variation. He emphasized that point, becau such information was really essential to enable the experience gained in project of such a nature to be utilized to the maximum advantage in th consideration of other similar problems.

A rough analysis of the bank section illustrated in Fig. 4, Plate indicated that for a shallow slide similar to those described in the Pap an average shear strength around the failure-surface of 7 lb. per square in would be required for stability, whilst for a deep slide passing down the Oldhaven beds about 11 lb. per square inch would be necessary. shear tests had been carried out on the material from the site the resul might have indicated whether the slope was on the verge of stabilit especially in respect of deep slides, or whether a substantial temporary lo of stability through hydrostatic pressures and so on, was responsible f failure.

In tests carried out at the Building Research Station on samples London Clay from various localities, values of between 7 and 15 lb. p square inch had been obtained for the shear strength, which were simil to those that he had calculated in connexion with the Paper.

**Mr. S. C. Lewis** observed that when he first went to Herne Bay, March 1936, in response to an urgent call, he was rather aghast at t spectacle presented by the condition of the sea-front. The retaining wal cafés, bathing-cabins, and shelters were in a terrible state, and slips we still continuing. While he stood opposite Queen's avenue, looking over t top of the cliffs, a mass of earth about a cubic yard in extent suddenly sl down the face of the cliff right under his feet, leaving a surface like a soa film, as if butter had been spread to assist its descent. He was able persuade the Council to have the bore-holes made, and he agreed with M

Carpmael that the boreholes indicated what was unnecessary rather than what had to be done.

Mr. T. H. Seaton wished to congratulate the public authorities of Herne Bay upon their wisdom in arranging for the problem to be considered by an experienced engineer, and the Author upon his foresight in carrying out the preliminary investigations described before the works were designed. Large sums of money had been spent unwisely in dealing with similar problems, only to aggravate the position. Whilst agreeing generally with the measures that had been adopted to stabilize the cliffs at Herne Bay, he would observe that his experience was that for the subsidiary drains a "chevron" system yielded better results than did a "herring-bone" system, especially in providing surface support to unstable material, and also that the pores of porous pipes at times became choked with silt. He had therefore used glazed earthenware pipes with open collar joints, which preserved the continuity of the pipe should the alignment be disturbed. Moreover, if a continual stream of water ran down the pipes, it was advisable to lay them on concrete; otherwise, sooner or later, the bed of the pipes would be washed away. He had known of many instances in which a channel had formed below the pipe and further trouble had been caused. He had also found that drainage trenches in clay, other than those used as counterforts, were best filled with flints, which did not become choked so quickly as did hardcore.

Works had been carried out recently on the London and North Eastern Railway in connexion with the stabilization of clay slopes. The single line between Thorpe-le-Soken and Clacton-on-Sea had been doubled, and at Great Holland, in a particularly unstable cutting, there were slips from end to end. The upper portion of the cutting was water-bearing gravel and sand, whilst the lower portion was clay. In widening the cutting the gravel portion was trimmed to a slope of 1 in 2, and the clay portion to a slope of 1 in 3. In some instances the old slips in the cutting went much deeper than what was to be the finished surface of the cutting, and those slips were dealt with by counterforts constructed in accordance with the recommendations made in a Paper which he had presented to The Institution[1]. Shallow slips and waterlogged and unstable surfaces were stabilized by means of trenches about 2 feet in width filled with hardcore, whilst pipes were laid where there was a continuous stream of water. The slopes were covered with a layer of ashes about 6 inches in thickness. At the foot of the cutting was laid an open-jointed earthenware pipe drain on concrete in a flint-filled trench, to carry away the water from the counterforts and drainage-trenches. The formation for the new line was covered with a blanket of fine ashes 12 inches in thickness. The cutting had now survived two winters, the second of which had been

---

[1] "Engineering Problems Associated with Clay, with Special Reference to Clay Slips". Journal Inst. C.E., vol. 8 (1937–38), p. 457 (April 1938).

particularly severe. No slips had yet occurred, and it appeared probable that the measures adopted would prove permanently successful. The measures taken were illustrated by lantern slides.

In a cutting on the Edgware branch line of the London and North Eastern Railway, trouble from slips had been experienced for many years. A rough wall of slag had held up the cutting for some time, but when the line was widened recently the old slips started to move. The problem, which was one of the most difficult that he had ever tackled, was dealt with successfully by constructing vertical trenches, 6 feet wide and about 18 feet apart, packed with hardcore, as counterforts from top to bottom of the cutting. The work was illustrated by lantern-slides.

**Mr. M. F. Barbey** had had to deal with a slip in a railway cutting in material rather similar to the clay at Herne Bay. He had brought a sample with him, in the hope that the Author might be able to confirm some of the values he had obtained for it. That sample had a moisture-content of 30·2 per cent., a plastic limit of 23·0 per cent., a liquid limit of 74 per cent., a cohesion of about 3·3 lb. per square inch, and a density of about 110 lb. per cubic foot. The cutting was about 30 feet high, and the clay exhibited a tendency to disintegrate into small pieces similar to that of the clay at Herne Bay. It was very difficult to maintain the bank at a slope of 1 in 2, or even of 1 in 3.

He thought that the trouble at Herne Bay was chiefly a series of surface slips, due to disintegration on account of weathering; and the only treatment for such slips was to make a series of small drains and to seed the surface, as had been done. Two areas, however, were marked in Fig. 3, Plate 1, where underground water occurred, and where probably much more deeply-seated slides had taken place. From Fig. 4, Plate 2, he had endeavoured to test that out on the circular-slip theory, and had run an arc from the top crack which appeared in the upper inspection pit, to just below the ordnance-datum line, where it struck borehole No. 6; it cut borehole No. 7 at the junction between dark brown clay and blue clay, which was 10 or 15 feet below the ordnance-datum line, and it came out about 40 feet from the piling. Perhaps the Author could confirm that that was where the blue clay did appear. An approximate analysis gave a value for the shear-strength of 4·62 lb. per square inch for that particular circle of failure.

In some works carried out so long ago as 1844, on the down side of the line to the north of Euston station, shallow drains had been put in on the surface of the cutting, and were reported at the time to have been successful. In that case specially-made pipes, with a series of tapered holes in them, had been employed.

He had adopted a toe-wall in the case of small banks, and had found it fairly effective, but it was probably not so efficient as the counterfort type of drain, although much cheaper.

**Mr. J. M. Lacey** remarked that in a Paper which he had presented to

The Institution some years ago, he had dealt with the erosion of the Beltinge cliffs at Herne Bay[1]. He exhibited a lantern-slide showing the surface of the Blean, which was an outlier of the lowest part of the London Clay formation, about 16 miles long and about 6 miles wide. Patches of gravel and brickearth which were visible were possibly the remains of glacial or post-glacial floodings due to the melting of snow on the chalk downs, scouring out ravines that became filled with gravel, sand, and brickearth when milder conditions prevailed. The area in question was, perhaps, the remains of some ancient river system having its origin in the Wealden Anticlinal, of which the brooks of Swale cliff, Hampton, and the Oldhaven gap were the small surviving remnants. The district was rich in relics of the Pleistocene and post-glacial periods, and several people in the town had discovered mammoth tusks in the debris from the cliff. Considerable quantities of ground-water were present in the area, the water-bearing beds varying according to the hollows scoured in the original London Clay surface. The manager of the Herne Bay waterworks had told him that, in laying the main to Beltinge, he had come across bands of gravel, 6 feet or more wide, full of water, and he had also experienced difficulty with water in the foundations of the service reservoir, situated on the highest point of the downs, and in the foundations of the new cottage hospital, which was $\frac{1}{4}$ mile east of the service reservoir. It was a curious fact that in some areas large quantities of water were found, whilst in other areas there was no water at all.

The water collected in the hollows of the London Clay did much to destroy the equilibrium of the cliffs, and large masses of the cliffs had slipped and sunk with a marked inward dip towards the cliff-face. Large landslides, rather than slips, had taken place. In 1896 an extensive landslide occurred in Queen's avenue, at the place where nearly $2\frac{1}{2}$ years ago about an acre subsided. The area affected by that slip included a footpath, a quantity of bush growth, and a large part of a field of barley. In 1903, a large landslide occurred, hedges and ditches remaining in place but sinking lower down. In January 1930, a large landslide occurred opposite the Rand, in which the land sank about 40 feet, carrying away garden-fences and hedges. All of these slips occurred where there had been previous slips, indicating that some weakness existed in the area.

It was obvious that in dealing with landslides of such a nature a retaining wall would be of little use, as it would be pushed aside, and that the real remedy lay in draining the water away from the cliffs.

The area where the works described by the Author were carried out had always been a source of trouble, owing to surface slips. The upper promenade had frequently been damaged. Before the construction of

[1] "Littoral Drift along the North-East Coast of Kent, and the Erosion of the Beltinge Cliffs near Herne Bay." Selected Engineering Paper No. 72, The Inst. C.E., 1929.

the concrete retaining wall and the bathing-cabins referred to in the Paper a timber sheet-piling retaining wall, which allowed some escape of water from the cliff slope, had existed, but even then slips frequently occurred, and farther down, eastward of the "Hundred Steps," gaps were left in the timber retaining wall in order to allow the passage of such slips. The construction of a solid retaining wall, with perhaps no allowance for drainage, had resulted in the catastrophic damage described in the Paper.

With regard to the boreholes, the London Clay in the area in question was the lowest part of the formation and was a dark-blue dense substance. The brown and mottled clay with sand was, he thought, of a later formation. Dr. S. W. Wooldridge had stated:[1] "The Blean is covered with gravel to a considerable extent of deposits of widely different ages. They seem to be best interpreted as the product of torrents due to the melting of snows on the chalk downs after the main glacial episode." East of Sea View road, beyond the houses, there was an excavation for a brickfield; possibly most of the mottled clay was not quite London Clay.

The slip where the works described in the Paper had been carried out was more in the nature of a surface slip than a great landslide, such as the landslides that he had described.

Why had the Author not carried the primary drains right through the retaining wall to the sea-face, dividing the wall into sections and providing a bridge over the cutting? The longitudinal drain that the Author had made was likely to become choked, and that might very possibly result in a further disaster similar to that described in the Paper.

**Mr. H. W. Holmes** observed that the district with which he was concerned—namely, Frinton-on-Sea and Walton-on-the-Naze—had cliffs which were similar to those at Herne Bay, and he had followed both the Paper and the discussion thereon with interest. On the basis of 10 years' experience of the type of cliff in question, he thought that the Author had undoubtedly gone the right way to work. Perhaps the Author was a little fortunate in that the cliffs at Herne Bay had assumed already a slope which was much flatter than that in the Frinton district, where the greensward was a very valuable piece of ground which the Council did not wish to lose. He had experienced the greatest difficulty in carrying out cliff-stabilization works, because there was a vertical face at the cliff-top where the slips always occurred. He had found no difficulty in dealing with the lower half of the cliff, where it was merely a question of getting rid of the brown clay which had previously slipped, leaving only sufficient to prevent the blue clay from exposure to the air and consequent deterioration. The difficulty lay in dealing with the upper 30 feet, where the blue clay was overlain by the brown clay, and where an attempt had to be made to retain the cliff-edge. That was a very difficult problem, and

[1] "The 200-foot Platform in the London Basin", Proceedings Geologists Association, vol. xxxix (1928, part I), p. 1 (25 April 1928).

he would welcome any assistance that could be given to him in dealing with it. It was easy to cut back the ground to a slope, but expensive artificial works, such as were required to retain the cliff-edge in its present position, were not, he thought, practicable for his Council. The district had a total sea-frontage of 6½ miles, 3 miles of which was owned by the Council, including 2 miles affected by slips. Taking the cost of the works at Herne Bay as a basis (he thought it was a very good basis and probably the minimum cost), the cost of such works in his district would be almost £190,000 ; and as the rateable value of the district was £110,000 it was obvious that any attempt to have such works carried out would not meet with much success.

**The Author,** in reply, observed that several speakers had asked if any quantitative information were available with regard to the clay extracted from the boreholes. No samples were actually tested in the laboratory. The boreholes were sunk 4 years ago, which, as far as he was aware, was before the present technique evolved by the Soil Mechanics Section of the Building Research Station had been developed.

Sample cores were extracted from most of the boreholes at intervals of about 5 feet, and carefully examined at the surface, and, in addition, a complete record was kept of the behaviour of the holes and of their tendency to swell during sinking, so that, short of taking laboratory tests of the moisture-content and strength of the clay, the engineers acquired a fairly intimate knowledge of the subsoil, all of which was carefully recorded at the time.

The Author agreed, however, with Mr. Wentworth-Sheilds, that that information allowed only a general comparison to be made between the samples extracted from different places and depths, whereas a comparison on a quantitative basis of the physical properties of the underlying clay before and after the completion of the surface drainage-works would have provided a useful basis for the design of further works of a similar nature.

Mr. Wentworth-Sheilds had asked why the "herring-bone" laterals were required to be not less than 4 feet deep. It was considered just as essential for the laterals, as for the primaries, to be carried down to the undisturbed clay below the slides. To have bottomed up those trenches with 2 or 3 feet of slurry below the invert would have invited further movement. It was realized also that the primary, lateral, and interceptor drains would have a combined buttressing effect upon the cliff-face as a whole. The support afforded by shallow surface-drains would have been negligible, but by carrying the inverts of all drainage trenches below the top of the undisturbed clay, each drainage system was keyed into the solid, and the slides were broken up into a series of small independent units.

Mr. Ellson had suggested that reference-points should be fixed in the solid below the slurry as a check on the equilibrium of the mass, but the Author was of the opinion that an equally satisfactory check could be obtained by keeping a careful watch on the behaviour of the brick pavilions and concrete retaining walls at the foot of the cliff. Any further movement

of the underlying clay would be at once detected by the formation of cracks in the masonry, as had, in fact, occurred before the drainage-works were commenced, whilst further movement of the surface layer would be detected by a tendency for the material to surge up above the top of the retaining walls. Neither of those movements had occurred.

In dealing with the Sevenoaks slip, Mr. Ellson had had the bottoms of the trenches paved with concrete before being filled with rubble. The Author had also considered the possibility of paving the primary drains with concrete, but it was found that that would have increased the cost of the drains by about 13*s*. per linear yard, and the proposal was, therefore, not proceeded with. There had been no signs of any movement in those drains since they were constructed, and concrete inverts would therefore have been an unnecessary refinement at Herne Bay.

The particulars of the large fall of chalk in the Folkestone Warren, furnished by Mr. Ellson, as well as the explanation of the manner in which those falls took place, were informative and interesting. Where a lack of stability in a chalk cliff constituted a potential source of danger to the public or to property, it was not unusual to strengthen the cliff with masonry buttresses, keyed into the face, sometimes arched over at their upper ends. In very exposed or dangerous situations it was occasionally found necessary to cover the entire face in concrete or masonry. The danger of a chalk slide was that usually there was no warning when it was about to occur, and the results might be catastrophic. The design of the supporting buttresses was a matter of judgment, or guesswork, because nothing was known of the magnitude, position, or direction of the outward thrust which would result from the separation of any further masses of chalk from the body of the cliff. The risk could, however, be appreciably reduced by cutting back the face to a batter of 4 to 1 before building the buttresses, and in cases where the land at the top of the cliff was valuable, the edge of the cliff could be restored to its former position by the construction of a beam-and-slab or arched decking resting on piers founded on the buttresses. The decking would have the advantage of protecting the top portion of the cliff against rain.

Sir Cyril Kirkpatrick had stressed the importance to the engineer of having a sufficient number of preliminary borings taken before he was called upon to complete his designs, and the Author agreed that authorities were sometimes reluctant to incur that additional expense. At Herne Bay they were lucky, and no objections were raised. As a result, the works were carried out substantially in accordance with the designs, and the council were saved the expense of claims arising from alterations made during the execution of the works.

The porous pipes, concerning which Sir Cyril Kirkpatrick had asked for further information, were of a proprietary make. The upper two-thirds of the periphery was of a highly-porous breeze concrete, and the invert was of ordinary non-porous mass concrete. Their chief drawback lay in the

difficulty in keeping the porous part clean while the pipes were lying about, or while they were being laid in the trenches. To keep the pipes clean called for constant and unremitting supervision. The pipes were 18 inches long and had ogee-type joints. They were laid with open joints, so that even if some of the pores became clogged during the laying, water was still free to percolate through the joints and the trench could not become water-logged.

The rainfall on the days on which the discharge from one of the outfalls had been measured, and on the preceding days, was as shown in Table II.

The sum of £2,700 0*s*. 0*d*. had been included in the contract under the heading of contingencies, and the reason for the slight excess of cost over estimate was that the council agreed to pay the contractors an additional rate for hauling the excavated material to a spoil site below Queen's avenue, instead of dumping it into the sea opposite the works.

TABLE II.

| Date. | Rainfall: inch. | Measured discharge: gallons per hour. |
|---|---|---|
| 24th November . . . . | 0·34 | — |
| 25th ,, . . . . . . | 0·37 | 600 |
| 15th December . . . . | — | — |
| 16th ,, . . . . . . | 0·16 | 110 |
| 2nd January . . . . . | — | — |
| 3rd ,, . . . . . . | 0·16 | 450 |
| 11th ,, . . . . . . | 0·05 | — |
| 12th ,, . . . . . . | — | 90 |
| 13th ,, . . . . . . | — | — |
| 14th ,, . . . . . . | 0·39 | 40 |

The Author was somewhat puzzled by Mr. Carpmael's remark that railway engineers were often compelled to execute work before they were in a position to investigate the cause of the trouble. If a slide occurred in a railway cutting the Author fully appreciated that the first and most urgent step to be taken was to clear away the slurry so as to get the road open to traffic as quickly as possible, but it did not take long to bring a light boring rig to the site, and boreholes could presumably be drilled at the same time as the more urgent remedial works were being carried out.

The boreholes at Herne Bay ruled out certain extravagant theories which had been advanced as to the cause of the slides, and, as Mr. Carpmael had stated, prevented needless expenditure on unnecessary measures. The junction between the slides and the undisturbed strata was in most cases clearly defined by a marked change in the appearance and consistency of the material, and the drains were invariably carried down to depths of not less than a foot below the base of the slides.

He was unable to agree with Mr. Carpmael as to the lack of support

afforded by the lateral drains. That those lateral drains played an essential part in stabilizing the slides was proved in a conclusive manner about 12 months after the main drainage-works were completed. It was found then that a further extensive slide on the undrained portion of the cliff just to the east of the Hundred Steps was having a destructive effect on the treated portion of the cliffs east of primary drain No. 1, and on the engineers' recommendation the council decided to have that slide stabilized by means of an additional primary drain and lateral drains. In an endeavour to economize, the engineers decided to provide lateral drains only on the east side of the new primary drain; that was to say, between the primary drain and the Hundred Steps, as it was hoped that the primary drain would have the effect of drying out and stabilizing the other half of the slip, and would obviate the need for further expenditure for some time to come.

The work was carried out, and gave entirely satisfactory results as far as the western half of the slide was concerned, but the other half continued to move, and masses of slurry began to pour over the rubble filling of the new primary drain, forcing the covers off the inspection-pits, and clogging the pores of the drain. It was, therefore, decided to balance up the system by constructing the other lateral drains between the primary drain and a comparatively stable bluff which lay 30 or 40 feet farther east. The result was entirely satisfactory, and left no doubt in the Author's mind as to the essential part played by the lateral drains in helping to bring about a state of equilibrium.

Mr. Cooling's analysis of the mechanism of clay slides laid particular stress on the occurrence and formation of fissures deep down in the body of the clay, and their responsibility for the disintegration of clay slopes. If the full hydrostatic pressure could be developed in a deep crack, the pressure at a depth of 24 feet below the surface would be 10½ lb. per square inch, and it would be appreciated that a pressure of that magnitude acting against an unsupported face would constitute a powerful disturbing force.

Mr. Seaton had mentioned that a "chevron" system yielded better results than did a "herring-bone" system of lateral drains, but if the "chevron" system were constructed as illustrated in *Fig. 2* of Mr. Seaton's Paper on clay slips * it was the same as the system adopted at Herne Bay, except that additional drains, known as "link" drains, were introduced, parallel to the primaries through the intersection of the laterals, in order to subdivide the slides further and to provide additional support to the upper ends of the laterals.

He noted that in Mr. Seaton's opinion the pores of porous pipes at times became choked with silt. It should be remembered that the pores extended over the upper two-thirds of the periphery, and were in contact with rubble

* "Engineering Problems Associated with Clay, with Special Reference to Clay Slips." Journal Inst. C.E., vol. 8 (1937–38), p. 466 (April 1938).

the body of the trench and not with the sides of the trench. In view of ıe fact that the rubble was covered with a filter layer of clinker below ound-level, the only way in which silt could be carried into the drains ould be by being washed down the sides of the trench and across the oping invert towards the pipes. Admittedly there might be a tendency ter a time for the lower edge of the porous shell to become choked, but ater was still free to find its way into the pipes through the open joints. he possibility of channels being formed below the inverts of the pipes was rovided for by constructing drainage cavities against the upstream faces f the inspection-pits and cutting weep-holes through the walls, as shown *Fig. 7* (p. 422, *ante*).

The Author noted that Mr. Seaton preferred flints to hardcore. Flints ere quoted for at 5*s*. 6*d*. per cubic yard extra, and would have involved an dditional expenditure of approximately £5,000 on the contract. The only isadvantage of brick rubble, so far as he could see, was that certain con- gnments contained an excessive amount of dust, but most of that was parated out when the rubble was loaded by hand into skips ready for wering down the face of the cliff.

In reply to Mr. Barbey, the portion of the foreshore where an upheaval f blue clay was alleged to have occurred prior to the commencement of the orks was opposite the section shown in Fig. 4, Plate 2, below the deposit f water-bearing sands and gravels revealed by boreholes Nos. 2, 3, and 4S.

With regard to Mr. Lacey's concluding remarks, the Author was aware f no reason why the lower longitudinal drain should become choked. It as carried well down into the solid clay below the slides, and was provided ith manholes from 50 to 70 feet apart which made the inspection of each ngth a comparatively easy matter. Discharge pipes led from the longi- udinal drain to the foreshore at convenient intervals.

Each of the terraces between the retaining wall and the lower promenade ad its own system of drains, and a large part of that area was paved in oncrete. It would, in the Author's opinion, have been a needless expense o have extended the primary drains seaward of the retaining wall. Between points A and B, in Fig. 3, Plate 1, where the retaining wall had een overturned, the primary drains were extended to the lower level by utting gaps through the debris, and a supplementary deep longitudinal rain was constructed along the back of the damaged bathing cabins.

The lower longitudinal drain was an essential part of the drainage cheme. It served the useful purpose of drying out the wet clay and lurry behind the retaining wall down to a depth well below the foundation, hereby reducing the thrust on the wall, quite apart from its function as a neans of conveying the drainage from the primaries to the outfall.

Mr. Holmes had referred to the clay cliffs at Frinton-on-Sea. If *Fig. 11* in the Discussion on Mr. Seaton's Paper * were a representative

* Journal Inst. C.E., vol. 8 (1937–38), facing p. 476 (April 1938).

view of those cliffs, they displayed a marked similarity to the cliffs at t eastern end of the site at Herne Bay as shown in *Fig. 2* (facing p. 41 *ante*), both in regard to their general appearance and their slope. T retention of the vertical face at the top was a difficult matter, because was there that the slides usually originated, starting with a crack a fe feet back from the face, followed by the breaking away of large lumps clay. The Author saw no alternative but to dress the face back to a slo which would stand a reasonable chance of remaining stable after bei drained.

Some form of sea-wall or breastwork along the toe would also necessary to prevent erosion by the sea, but it would be a waste of mon to build a breastwork without the drains if conditions were as bad as tho at Herne Bay.

## Correspondence.

**Mr. H. J. F. Gourley** observed that he had been asked, a few years ag to collaborate with Mr. B. L. Hurst, M. Inst. C.E., in reporting upon exte sive slips which had occurred at Frinton, with consequent bulging and lif ing of a reinforced-concrete retaining wall at the foot of the cliff. T materials involved were the basic blue clay with overlying plastic or so brown clays, within and near the top of which were beds and pockets waterbearing sand, and the appearance was very similar to that describe by the Author, namely, alternating shoulders and transverse valley It having been made clear that a thorough geological investigation was a essential preliminary to formulating recommendations, an extensive bori programme was carried out, so that over the whole area the surface of th blue clay was established and the extent, thickness, and relative pe meability of the sand beds were determined.

Samples of the clay were mechanically analysed: the brown clay averaged about 68 per cent. by weight of clay, 24 per cent. of water, an 8 per cent. of sand, most of which passed a 180-mesh sieve, whilst the blu clay contained 37 per cent. of clay, 24 per cent. of water, and 39 per cen of sand, most of which passed the 180 mesh. From those results, and fro the behaviour of the samples whilst they were being prepared for analysi it was concluded that the brown clays were of an unstable characte liable to break down and/or slip in the presence of water, and necessitatin in any event, flat slopes protected from rain and measures to prevent th access of water from below; the blue clay was regarded as of relativel high stability and considerable bearing capacity.

There appeared to be little doubt that the slips were essentially th result of water-lubrication, which increased the inherent instability the brown clay and caused it to slide on the blue clay, the surface of whic was more or less level near the top of the 75-foot high cliff, but had slightly undulating fall towards the foreshore of about 1 in 3, commenci some 200 feet back from the line of the toe-wall.

It was concluded that the only method of stabilizing the cliffs was to pick up all water in the sand by a master drain laid as far back as was practicable from the heads of the valleys, and at such a level as would be at all points below the bottom of the lowest sand-layers. That involved the laying of a 12-inch open-jointed concrete drain at a maximum depth of 18 feet; the trench was to be filled with gravel up to the top of the water-bearing sand or to 3 feet below the surface of the ground, whichever was the lower, and above that with a layer of ashes, and finally with ordinary refilling. Manholes at distances apart not exceeding 100 yards were to be constructed for rodding and clearing any sand which might otherwise have accumulated. It was not considered prudent to rely upon an outfall from that master drain laid down the cliff, even in the blue clay, and, instead, a vertical shaft was to be constructed from which an elliptical concrete tunnel 5 feet by 3 feet would lead to a manhole constructed almost entirely in blue clay behind the retaining wall. The drainage water was finally discharged on to the foreshore through a pipe laid in the blue clay and fitted with a tidal flap. That work constituted the first and most important stage; subsequently, and after the elimination of the water-lubrication had had time to be effective, the slopes were to be flattened to a hollow curve and covered with turf obtained from the adjacent meadows.

It was not considered that shallow surface drainage of the final slopes would be necessary if care were taken in the turfing. At Herne Bay the master drain extended over a greater length of frontage than was covered by the drains in the face of the cliff; had the undrained face shown any tendency to slip?

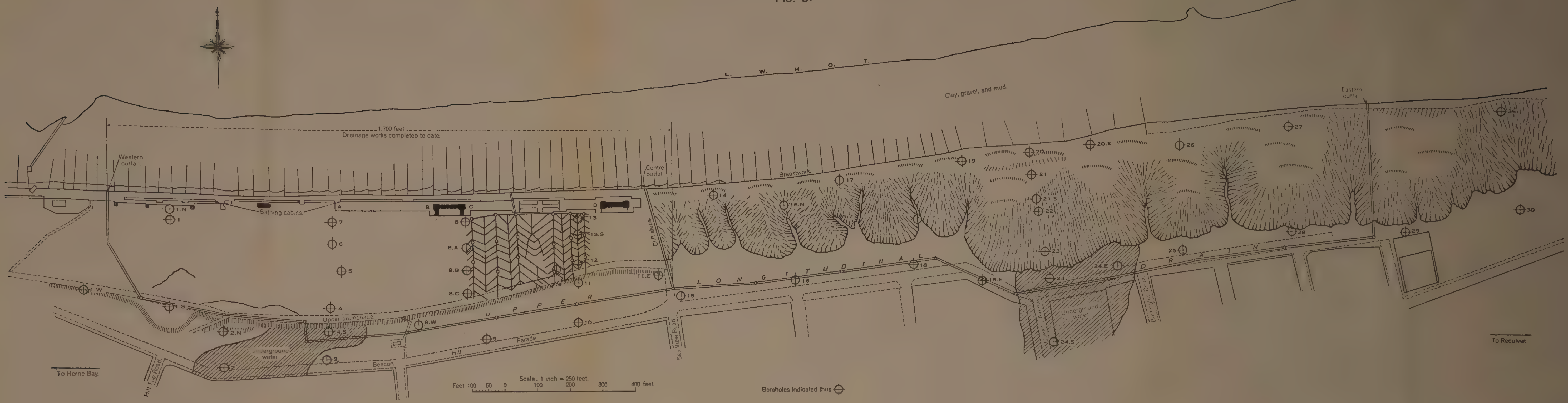
**The Author,** in reply, observed that from the description furnished by Mr. Gourley, it appeared that conditions at Frinton, so far as the water-bearing strata at the top of the cliff were concerned, were very similar to those at Herne Bay, and the measures which he proposed to adopt to arrest the flow of underground water towards the cliff-face were also very similar. The analysis of the clays at Frinton showed a surprisingly high sand-content. At Herne Bay, the upper longitudinal drain was 3,800 feet long, but only 1,800 feet of the cliff-face had been stabilized by drainage. Along the 2,000 feet of undrained cliff, movement continued to occur on a large scale. At Queens avenue, where the upper longitudinal drain passed through a deposit of water-bearing sand and gravel, the drain had had a retarding effect on the crumbling away of the cliff-top, but the bulk of the cliff-face was still quite unstable and would continue to be so until the Council were in a position to extend the surface-drainage of the cliff-face eastward along the remainder of the frontage. The only stable part of the cliff-face in that area was the spoil-dump below Queens avenue, which had had rubble drains built through it as the material was tipped, the top of the drain being kept level with the top of the filling.

Mr. Gourley had proposed to flatten the slopes at Frinton to a hollow

curve and cover them with turf and to omit surface-drainage. Th Author's experience of the Herne Bay cliffs led him to believe that stabilit could not be achieved solely by covering the slopes with turf. Troubl had been experienced at Herne Bay due to the persistent sliding of som quite moderate slopes, which were thickly carpeted with grass, and fa removed from the disturbing influence of percolation of ground-water fro above. Sliding had continued to take place until surface-drainage ha been introduced, since when no further movements had occurred. Par ticular care had been taken to do as little damage as possible to the tur while the drains were being excavated.

# CLIFF-STABILIZATION WORKS IN LONDON CLAY.

FIG: 3.

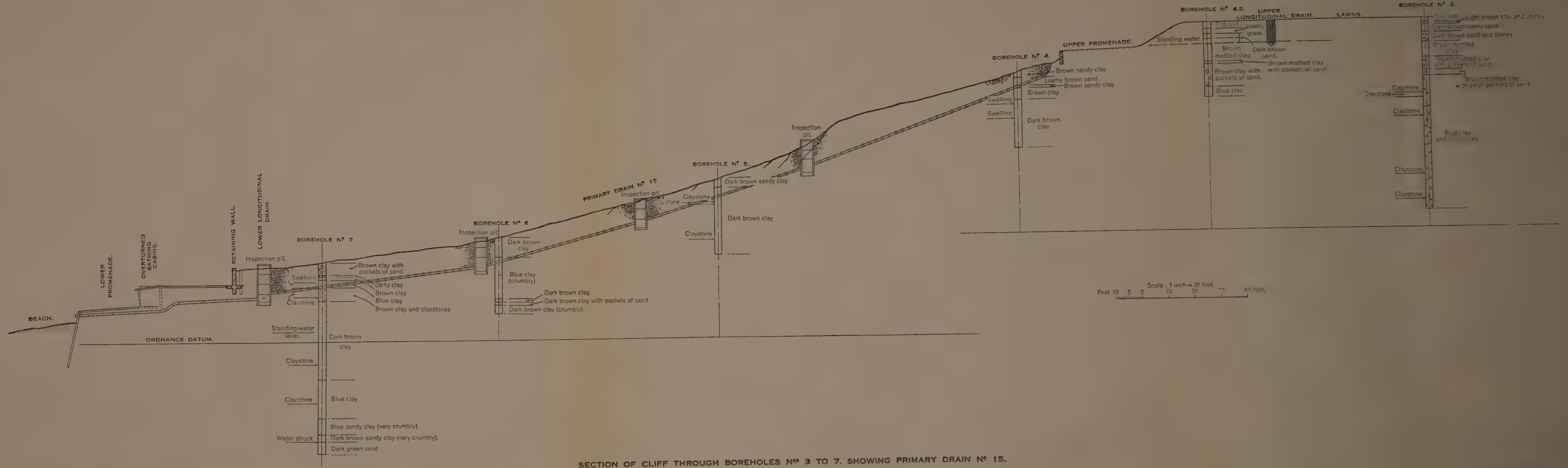



GENERAL PLAN OF THE EAST CLIFF. HERNE BAY.

WILLIAM CLOWES & SONS, LIMITED: LONDON.
The Institution of Civil Engineers. Journal. October, 1940.
J. DUVIVIER.

# CLIFF-STABILIZATION WORKS IN LONDON CLAY.

Fig: 4.




SECTION OF CLIFF THROUGH BOREHOLES Nos 3 TO 7. SHOWING PRIMARY DRAIN Nº 15.

J. DUVIVIER

William Clowes & Sons, Limited: London.

## EXTRA MEETING.

28 May 1940.

SIR CLEMENT DANIEL MAGGS HINDLEY, K.C.I.E., M.A., President, in the Chair.

## PRESENTATION OF THE JAMES ALFRED EWING MEDAL.

**The President** said that it afforded him great pleasure to present the James Alfred Ewing Medal for 1939 to Professor G. I. Taylor, Fellow and Yarrow Research Professor of the Royal Society and Fellow of Trinity College, Cambridge, who was distinguished for his development of various aspects of mathematical physics, many of which had found applications of great importance in the engineering field. His work on the problem of the turbulent fluid, with special relation to air-movements and tides, had furthered knowledge of meteorology, and had been applied with particular success to problems of aeronautics, being of material assistance to the fruitful use of the wind-tunnel. The practical character of Professor Taylor's researches was exemplified by his work on the torsional stiffness of airscrew-blades. His work had forged new links between science and engineering, to their mutual advantage.

The Medal, the President mentioned, was awarded to a person, whether a member of The Institution or not, for specially meritorious contributions to the science of engineering in the field of research.

## THE SIR CHARLES PARSONS MEMORIAL MEDAL.

**The President** stated that the Parsons Memorial Medal was to have been presented that evening to Dr. H. L. Guy, F.R.S., M. Inst. C.E., but that, owing to Dr. Guy having had to take immediate decisions consequent upon Government instructions, it was impossible for him to be present that evening.

The Sir Charles Parsons Memorial was founded in 1936, and comprised the erection of a memorial in Westminster Abbey, a Parsons Memorial Library, and an annual lecture on any of the subjects in which Sir Charles was interested, to be delivered by a distinguished man of any nationality, to whom a commemorative bronze medal was to be awarded. The lecturer was nominated in turn by certain Institutions, and Dr. Guy had been nominated as the Lecturer for 1939 by The Institution of Civil Engineers. His subject was "Some Researches on Steam-Turbine Nozzle-Efficiency."

The Lecture was not delivered in person owing to the war, but was printe in the December number of the Journal of The Institution.

The President mentioned that Dr. Guy had been Chief Engineer in th Mechanical Department of the Metropolitan Vickers Company since 191 and that he was a Whitworth Exhibitioner, Bayliss Prize-winner, an Thomas Hawksley Medallist, and was also a Vice-President of the Institu tion of Mechanical Engineers.

## JAMES FORREST LECTURE, 1940.

**The President** said that the James Forrest Lecture had been estab lished and endowed in honour of Mr. James Forrest, who had been Secretar of The Institution from 1859 to 1896, and Honorary Secretary from 189 until his death in 1917.

Mr. Forrest bequeathed to The Institution some pieces of silver plat which had been presented to him during the course of his life. Tha plate was normally exhibited on each occasion of the James Forres Lecture, but it would be appreciated that, owing to war conditions, i was undesirable that the plate should be displayed that evening.

He had great pleasure in introducing the Lecturer, Professor E. V Appleton, M.A., D.Sc., LL.D., F.R.S., Secretary of the Department o Scientific and Industrial Research, whose scientific achievements wer well known. Dr. Appleton had been formerly Professor of Physics in th University of London and Professor of Natural Philosophy at Cambridg University. He was a Hughes Medallist and a Fellow of the Roya Society. His activities also included service as Chairman of the Britis National Committee for Radio-Telegraphy and as President of the Inter national Scientific Radio Union, whilst in 1932 he was a Vice-Presiden of the American Institute of Radio Engineers.

# "New Materials for Old."

By Edward Victor Appleton, M.A., D.Sc., LL.D., F.R.S.

### Introduction.

It is a great honour to be asked to follow the many distinguished scientist who have previously lectured to The Institution under the auspices of th James Forrest Foundation. Two years ago, Sir Frank Smith, my pre decessor now in two capacities, selected, as the topic of his James Forres discourse[1], certain problems of the gaseous state of matter. He described for example, how the simple theory of an ideal gas as a conglomeration o molecules, influencing one another only by impact, had failed to accoun

[1] "Disorderly Molecules and Refrigerating Engineering." Journal Inst. C.E. vol. 9 (1937–38), p. 239 (June 1938).

for certain basic physical facts; and he pointed out that this failure had led to the conclusion that the gaseous molecules also exercise mutual attractive forces, called van der Waal's forces, which become of importance when the distance between molecular centres is small. We make use of these forces of attraction, which are exhibited by the reluctance of molecules to part company, in practical methods of attaining low temperatures.

In dealing with the subject of new materials I shall be obliged to restrict my survey in many ways, so vast is the field which invites consideration. I shall deal, for example, only with solid materials and so shall have occasion to consider briefly the forces which cause a solid body to retain its shape in the absence of external forces. It is necessary to inquire, in this connexion, whether the attractive forces between gaseous molecules, which Sir Frank dealt with 2 years ago, are sufficient to account for the cohesion of solids; to ask whether they completely represent, in fact, the mortar with which the molecular bricks of a solid stick together.

Now, whenever we use a material we do so because of some property or combination of properties which it possesses, and these may be mechanical, optical, electric, or magnetic in character. I think we are obliged to admit that, even when in possession of the many new materials now available, we cannot always find exactly what we would like. But at least we can claim that physical science is now sufficiently developed for us usually to be able to specify, with some precision, what it is that we would like, and that our ability to measure the great majority of physical properties enables us immediately to recognize such desired qualities when they are present. It would be a great exaggeration to claim generally that the physicist or chemist, armed with his present knowledge of molecular structure, can sit down and formulate, *ab initio*, the constitution of a material with specified characteristics. We can claim that, in certain respects, intentional synthesis of this kind is either possible or nearly so, but there still remains plenty of scope and opportunity for empirical adventures. I confess that it is a matter of no great personal regret that this should still be so.

It has often been said that mankind has progressed from the Stone Age, through the Bronze and Iron Ages, to the Steel Age. But if we designate any age by the more common materials used by man, it is clear that what has been called the Steel Age is now being succeeded by the era of plastics and alloys, about both of which I propose to say something this evening.

## Plastic Materials.

I do not think that any development in the field of new materials has excited a more lively interest than the growth of our knowledge of those industrially important organic materials known as plastics. This is doubtless due to the fact that the chemical technologist has here not been content with making mixtures of known substances, but has succeeded

in synthesizing entirely new compounds which are daily finding new uses and applications. Moreover, the scientific study of the relationship between chemical constitution and physical properties in this field has proceeded so successfully that progress in developing new materials has ceased to be entirely empirical and is becoming scientifically informed.

The term "plastic" is usually applied to substances which, during their fabrication into finished products, possess the property of plasticity. If deformed by mechanical stress, they assume a shape which is retained when the stress is removed. It is evident that such plasticity is only desirable under the temporary conditions of manufacture, since rigidity is required in the finished product. This rigid attainment of the desired shape is secured differently in the cases of the two different classes into which plastic materials may be divided. The thermoplastic class comprises products which at room temperatures are hard, but which, like ordinary paraffin wax, soften on the application of heat. On cooling they regain rigidity, and so can be shaped and reshaped at will. The thermo-setting or thermo-hardening class, on the other hand, consists of materials which, once formed by heat, become permanently hard and infusible, and which, if reheated, decompose before they soften. Their properties are thus analogous to those of china-clay, which is plastic and can be easily moulded before firing. During the firing a new constitution results which cannot be made to revert to the plastic form.

Plastic materials may also be classified according to their origin. In this connexion we may distinguish between natural, semi-synthetic, and wholly synthetic materials, as shown in Table I.[1]

TABLE I.—CLASSIFICATION OF PLASTICS.

| Natural. | Semi-synthetic. | Synthetic. |
|---|---|---|
| Shellac. | Casein-formaldehyde. | Coumarone-indene. |
| Rosin. | Cellulosic derivatives :— | Phenol-formaldehyde. |
| Amber. | Nitrate. | Urea-formaldehyde. |
| Bitumen. | Acetate. | Aniline-formaldehyde. |
| | Acetobutyrate. | Glycerol-phthalic anhydride. |
| | Benzyl. | Vinyl chloride. |
| | Ethyl. | ,, alcohol. |
| | Modified rubbers :— | ,, ethers. |
| | Chlorinated. | ,, esters. |
| | | ,, hydrocarbons (*e.g.* styrene). |
| | | Acrylonitrile. |
| | | Acrylic acids. |
| | | ,, esters. |
| | | Olefine-polysulphide and other synthetic rubbers. |
| | | Toluene sulphonamideformaldehyde. |
| | | Chlorinated diphenyl. |
| | | Polyethylene. |

[1] N. J. L. Megson and K. W. Pepper. "Plastics and Coal," *Chemistry and Industry*, vol. 59 (1940), p. 247 (13 Apr. 1940).

All such materials consist of amorphous mixtures of complicated and giant chemical molecules which do not form a regular crystalline architecture in the rigid state. They show no true solidifying-point, and so they may well be included in the group of super-cooled liquids, of which glass is perhaps the best-known example.

For the most part I shall ignore the natural plastics and concentrate on the synthetic products, since it is here that the most significant advances have been, or are likely to be, made in deriving intentional, as opposed to empirical, combinations.

The giant molecules I have mentioned may be either chain-like or web-like in character. A chain-like molecule may be generally represented by the formula $X-[R]_n-Y$, where X, R, and Y represent chemical groups, one of which, R, is repeated $n$ times. In the case of polystyrene, a new substance possessing very desirable insulating properties, the constitution may be represented by the formula

$$\begin{array}{c}C_6H_5\\|\\-CH-CH_2-\end{array}\left[\begin{array}{c}C_6H_5\\|\\CH-CH_2\end{array}\right]_n\begin{array}{c}C_6H_5\\|\\-CH-CH_2-\end{array}$$

where $n$ may be as large as 5,000. In this simple case X, Y, and R are identical (styrene) groups, though in other examples of chain compounds this need not be so.

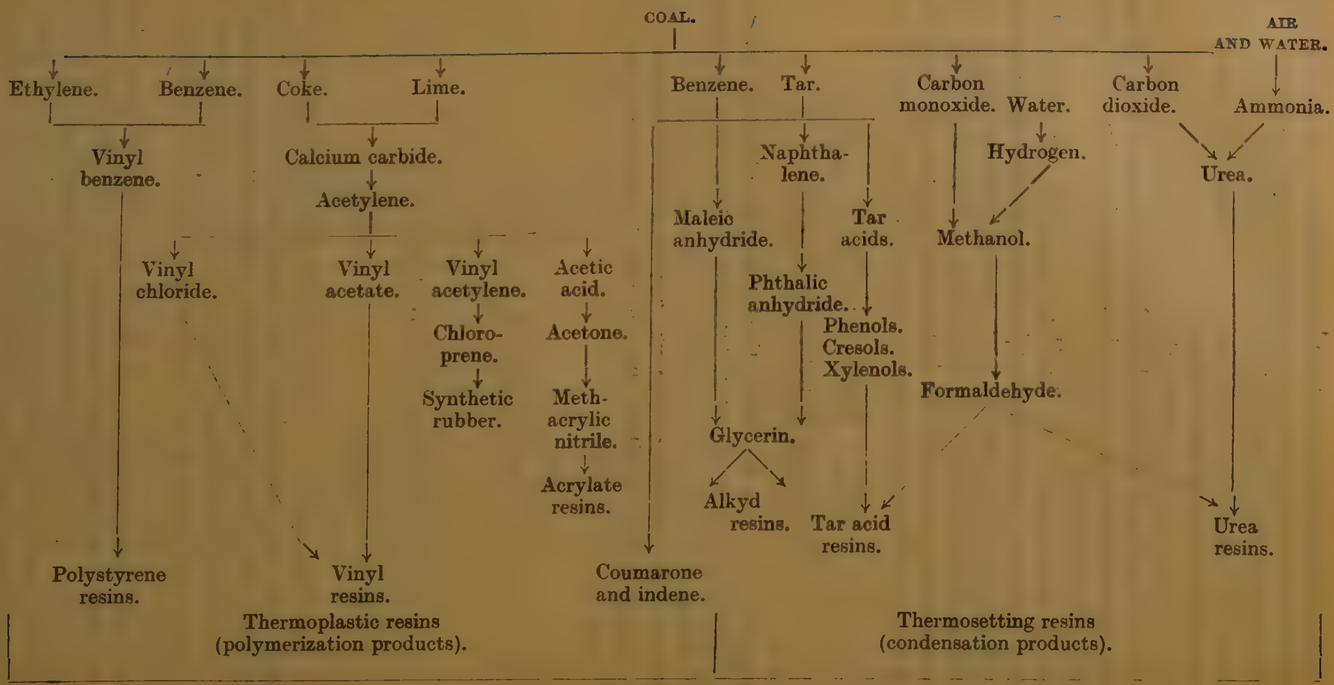
The same basic groups of styrene may also be incorporated with *p*-divinyl benzene as cross-links to form a web-like structure, the formula of which may be represented by

$$\begin{array}{cccccc} C_6H_5 & & & & C_6H_5 & \\ | & & & & | & \\ -CH & -CH_2- & CH & -CH_2- & CH & -CH_2- \\ & & | & & & \\ & & C_6H_4 & & & \\ & & | & & & \\ -CH & -CH_2- & CH & -CH_2- & CH & -CH_2- \\ | & & & & | & \\ C_6H_5 & & & & C_6H_5 & \end{array}$$

Generally speaking, substances with chain-like molecular structures are of the thermoplastic class, whilst those made up of web-like molecules belong to the thermo-setting class. Cross-linked thermo-setting materials have usually mechanical properties superior to those of thermo-plastics, but this is not the case as regards shock-resistance, in which the thermoplastics are superior.

Before mentioning a selection of the diverse applications of plastics, I wish to draw attention to the important fact that the raw materials from which many of the more commonly-used plastics are made can all be said to be directly derived from coal and its carbonization products. This fact is well illustrated by the family tree shown in *Fig. 1* (p. 452, *post*).

*Fig. 1.*




THE DERIVATION OF CERTAIN SYNTHETIC RESINS.[1]

[1] N. J. L. Megson and K. W. Pepper (*Chemistry and Industry*, vol. 59 (1940), p. 247 (13 Apr. 1940).

From the constructional standpoint the thermo-setting class of plastics is the more important. In this connexion it is of interest to recall the pioneer work of Dr. L. H. Baekeland,[1] who in 1908 discovered that, by heating phenol and formalin in the presence of an alkali, a resin was produced which, under the combined influence of heat and pressure, passed over into an insoluble and infusible form.

One method adopted by Dr. Baekeland was to mix the unhardened resin with a so-called " filler " of sawdust or wood-meal. Under the heat- and pressure-treatment the resin became plastic, filled the interstices of the mould, and then ultimately passed over to the hard infusible state. In another process the resin was spread on to paper, canvas, or cotton sheets, and the impregnated materials were bonded under heat and pressure to form the final laminated product.

Moulded products made in this way are now extensively used for electrical fittings of all kinds, for loud-speaker and wireless-receiver cases, door-handles, ash-trays, toys, and fancy goods. The laminated products in which the thermo-setting resinoid is the bonding agent have also now found many engineering applications. The combination of 50 per cent. of thermo-setting resinoid with 50 per cent. of fabric or paper reinforcement provides a material which has a specific gravity of 1·36, with an ultimate tensile strength (4·5 to 9 tons per square inch) as high as that of aluminium and some of its alloys.

From the engineering point of view perhaps the most spectacular present-day application of this type of plastic combination is as a bearing material in metal-rolling mills, the power-consumption being 20–50 per cent. less than when brass or bronze bearings are used. The secret of this efficiency lies in the fact that such rolling-mill bearings are lubricated and cooled with water. The coefficient of friction of the laminated material when water-lubricated is about one-quarter that of brass when similarly lubricated. Gear-wheels made of the same type of material are also finding increasing uses, being made in all sizes and degrees of accuracy, from clock-wheels to heavy mill-drives.

Much attention has recently been given to the problem of improving the mechanical properties of reinforced plastics with a view to their adoption as stressed members in aircraft. Using fibre fillings, laid uni-axially, tensile strengths exceeding 20 tons per square inch in the longitudinal direction of the fibre have been obtained, whilst the compression strength in the same direction is 11 tons per square inch. These figures are really remarkable for a material with a density only half that of duralumin and one-sixth that of steel. It cannot, however, be said that such materials have yet come into general use in aircraft construction, though much use is now made in wooden aircraft of plywood in which the older organic glues are replaced by synthetic glues of the phenol or

[1] British Letters Patent, Nos. 1921 and 1922 (1908).

urea-formaldehyde type. The great advantages of these synthetic cements are that the laminated product is both stronger and more water-proof, and the parting of the plies does not occur in damp conditions.

Transparent materials which do not fracture on impact have obviously a wide field of application. Certain thermoplastic substances, such as cellulose acetate and the acrylic resins, are satisfactory in this respect and are now almost universally used in aircraft for wind-screens and cabin windows, as well as in the production of moulded lenses. A further topical use of cellulose acetate plastics is in the eyepieces of the gas-masks with which we were all presented some time ago.

Quite an unexpected use for synthetic resins has followed from the discovery,[1] in the Chemical Research Laboratory of the Department of Scientific and Industrial Research, that some of them will remove basic radicals from solutions, whilst others remove acid radicals. Resins suitably chosen may therefore be used either to soften water, on the base-exchange principle, or, using a combination of resins in series, to render even sea-water potable.

Another example of the application of the use of synthetic resins is in connexion with the production of "creaseless" cotton. It has been known now for some time, thanks to modern methods of analysis, that cotton fibres consist mainly of cellulose molecules which exist as long chain compounds. As a result of this structure, the material is relatively strong for longitudinal forces, but can be easily bent. Moreover, it displays very little tendency to regain its shape when bent. In other words, it is easily creased. Since careful examination showed that the cotton fibre itself is of a porous nature, it was realized that if the air-spaces could be filled with some elastic substance, such as a synthetic resin, the creasing properties of the finished material would be greatly reduced. The way in which the desired result was achieved is of particular interest, for it was found that the giant molecules of the final resinous product were too large to enter the air-spaces, and it was necessary to impregnate the material with the primary substance of which the links of the chain are composed and to polymerize it, thus forming the chain molecules, in situ.

### *Some further Examples of Chemical Synthesis.*

Much progress in recent years can be recorded in the scientific study of the constitution of fundamental raw materials such as rubber, cotton, linen, wool, and silk, and in all cases it has been found that the basic molecules are of considerable complexity. It is true that some difference of opinion exists, for example, about the exact nature of the rubber molecule, but it is at least generally accepted that it is of chain form, a

[1] B. A. Adams and E. L. Holmes. "Absorptive Properties of Synthetic Resins." J. Soc. Chem. Ind., vol. 54 (1935), p. 1.

chain-length of about 12,000 Å being associated with a molecular weight of about 180,000. The synthesis of a material strictly identical with natural rubber in every chemical and physical respect is still unrealized by the chemist, but several products with properties closely resembling those of natural rubber, and for some purposes with better properties, are now commercial realities. The most important methods of making synthetic rubbers depend ultimately on coal, from which is derived acetylene, the parent substance in the family tree shown in *Fig. 1* (p. 452, *ante*). From acetylene both butadiene and chloroprene can be formed as shown in Fig. 2.

*Fig. 2.*

$CH\vdots CH$
acetylene

$CH_3\cdot CH(OH)\cdot CH_2\cdot CHO$
aldol
↓
$CH_2{:}CH\cdot CH{:}CH_2$
butadiene

$CH_2{:}CH\cdot C\vdots CH$
vinyl acetylene
↓
$CH_2{:}CH\cdot CCl{:}CH_2$
chloroprene

The polymerization products of both chloroprene and butadiene are rubber-like materials known commercially as neoprene and buna respectively. These products of chemical synthesis are superior in some respects to natural rubber. This is illustrated by the comparison set out in Table II, from which it is seen that, whilst neoprene and rubber

Table II.

| | Rubber. | Neoprene. |
|---|---|---|
| Tensile strength: kilograms per square centimetre | 216 | 224 |
| Elongation at break: per cent. | 625 | 602 |
| Resilience: per cent. | 63–65 | 63–65 |
| Abrasion loss | 0·110 | 0·110 |
| Percentage swelling at 70° C.— | | |
| in diesel oil | 480 | 58 |
| in transformer oil | 275 | 15 |

have very similar mechanical properties, the former is much superior to the latter in resistance to the deleterious effects of oil. Neoprene is also superior to rubber in resisting weathering and the effects of sunlight and ozone.

Cotton and linen have, as their base, molecules of cellulose, whilst wool and silk, being derived from animal tissue, consist largely of proteins. All four substances can be spun because, in each case, the constituent molecules are of the chain variety. Artificial silk made from cellulose, not originally derived from cotton and linen, is, of course, well known and needs no further mention here. Recently another form of artificial silk, called nylon, has been made entirely synthetically, and has a protein structure. This substance has the remarkable property that threads made of it can be cold-drawn to several times their original length, after which they become exceedingly strong and elastic, superior, in fact, in these respects to natural silk. The long molecules during the drawing process become oriented parallel to each other.

Nylon at the moment is dearer to produce than rayon, but is probably cheaper than natural silk. It has a good appearance, can be given a lustrous or mat finish, and can be dyed in a wide variety of colours, whilst owing to its high tensile strength and elasticity it has wearing qualities above the normal. Scientific research has, in fact, produced something better than the corresponding natural material.

*The Theoretical Tensile Strength of Synthetic Materials.*

In Sir Isaac Newton's treatise on "Opticks" there occurs these words: "The Parts of all homogeneal hard Bodies which fully touch one another, stick together very strongly. . . . There are therefore Agents in Nature able to make the Particles of Bodies stick together by very strong Attractions. And it is the Business of experimental Philosophy to find them out." The task set to natural philosophers by Sir Isaac has proved indeed a difficult one, for only now, two hundred years later, is the nature of cohesive forces beginning to be clearly understood.

The modern theory of cohesion is essentially an electrical one, for it can be easily shown that the electrostatic forces between the component atoms of a molecule, and between neighbouring molecules in a solid structure, are very much greater than the gravitational forces between adjacent particles. Quantitative methods have now been developed by the theoretical physicist for calculating the tension strength and Young's modulus of the molecule itself, assuming its component parts to be held together by what the chemist calls primary bonds. An example of this type of bond is the homopolar bond between two neutral atoms, where the so-called valence electrons form a pair and belong to both atomic nuclei in common. The C–C bond and the C–H bond, which are typical examples of the linkages in synthetic resins, are examples of this type.

In addition to the primary bonds between the components of a molecule, the chemist recognizes secondary bonds, one example of which is presented by the van der Waal's forces between neighbouring molecules. As was mentioned earlier, the existence of these forces was first recognized

in the study of gases, where the molecules are relatively far apart. The recent development of the theory of this effect showed that the van der Waal's force between two atomic systems is proportional to $\frac{1}{r^7}$ where $r$ denotes the distance between the systems. Thus it can be seen that these forces are relatively much greater in solids than in gases at ordinary temperatures and pressures.

Since quantitative theories are now available for the consideration of both primary and secondary bonds, it is possible to calculate the theoretical tensile strengths and Young's moduli for substances of known composition and structure, and to compare the values so obtained with experiment. Calculations of this kind have been carried out for typical synthetic resins possessing web-like structures by J. H. de Boer,[1] the results of which are shown, together with experimental values, in Table III.

TABLE III.

| Substance. | Tensile strength: tons per square inch. | | |
|---|---|---|---|
| | Calculated on primary valency basis. | Calculated on van der Waal's force basis. | Observed. |
| Phenol-formaldehyde . . | 2,700 | 25 | 5·0 |
| Cresol-formaldehyde . . | 2,400 | 23·5 | 2·4 |
| | Young's modulus: tons per square inch. | | |
| Phenol-formaldehyde . . | 7,000 | 28·5 | 377 |

It will be seen that, whether we consider the tensile strength to arise either from primary or secondary bonds alone, the experimental value is much lower than that calculated theoretically. It is curious that, on the other hand, the experimental value of Young's modulus is intermediate between that calculated on the basis of valency and van der Waal's forces.

The discrepancies between theoretical and experimental values of the tensile strengths of these thermo-setting resins is usually accounted for in terms of a theory proposed by A. Smekal [2] to account for similar discrepancies in crystalline bodies. According to this theory flaws or *Lockerstellen* are present in the substance and, at such weakened points, stresses are concentrated and rupture starts. From a chemical point of view such *Lockerstellen* are likely to occur, since it is improbable that macro-

[1] "The Phenomena of Polymerisation and Condensation." Faraday Society, London, 1935, p. 11.
[2] *Handbuch der Physik*, 24 (II), Berlin, 1933.

molecules are produced possessing all the theoretical linkages. Many reactive groups must be prevented from linking by steric conditions.

## Some New Metallic Materials.

For materials of the maximum available strengths we still have, of course, to rely upon metallic substances, though here again there is the same type of remarkable discrepancy between the experimental values of yield-stress and those calculated according to current theories of metallic structure.[1] An estimate of the strength of a metallic crystal can be made by a number of theoretical methods, but the values obtained are always greatly in excess of those measured in practice. It may be calculated, for example, that the maximum shear stress for a copper crystal should be about 400 tons per square inch, whereas the ordinary polycrystalline form of that metal has a yield-stress of about 6 tons per square inch, whilst single crystals of the same material undergo plastic deformation under stresses as low as 0·1 ton per square inch. It is clear from the last-mentioned figure that purity and regularity of atomic structure are not to be associated with good mechanical performance, and, indeed, the recent developments of metallurgical science show that high tensile strengths are most readily attained in metallic systems which are not simple, but composite, and in which the atomic grouping is not perfectly regular, but markedly distorted.

The influence of lattice distortion is well illustrated by the effects of cold-working. If an annealed metal is worked below its re-crystallization temperature, its mechanical strength and hardness increase with the amount of plastic deformation. The strength of a single crystal of copper may, for example, be increased to fifty times its initial value when its elongation attains a value of 50 per cent. X-ray examination of such cold-worked material reveals that the increased strength is to be associated with a marked distortion of the metallic space-lattice.

Such marked lattice distortion can very readily be effected in composite (that is, alloy) systems, where use is made of the variation with temperature of the solubility of one of the components in another. In the operation of age-hardening processes, which are much used in the light-alloy industry, a supersaturated solid solution of one metal in another is allowed, under controlled conditions, to precipitate the excess material in solution. As the very finely-divided precipitate is rejected, it leaves the parent lattice in a highly-strained and disturbed condition. A good example of the precipitation-alloy of this class is that of copper containing 2·5 per cent. of beryllium. In the fully-hardened state, the hardness of this material is about 380 on the Brinell scale, whereas the hardness of annealed copper

[1] R. Fürth, correspondence on the relation between melting and breaking. *Nature*, vol. 145, No. 3,680 (1940), p. 741 (May 11, 1940).

only about 40. At the same time, the ultimate tensile strength of the lloy is found to be of the order of 80 tons per square inch, which may be ompared with the corresponding value for cold-worked pure copper of 3 tons per square inch.

Considerable progress is also to be recorded in the development of aterials for cutting-tools and dies. I select for mention in this connexion he range of sintered metallic materials, for which the process of manu-acture is somewhat analogous to that employed in the case of ceramics. uch materials are prepared from ingredients in the form of fine powders, hich are pressed to shape and subsequently heated to a sufficiently high emperature for bonding of the individual grains to take place. Since omplete melting is not necessary, this process is particularly suited to the abrication of high-melting-point metals and metallic carbides. A very nportant development of such sintered materials is the production of utting-tool tips from tungsten carbide, titanium carbide, and cobalt, he cobalt acting in this case as a bond. The employment of such tool-ips has permitted the acceleration of a large variety of machining perations. Owing to the high melting-point of the carbides and the elatively small amount of bonding material, there is very little thermal oftening at the cutting-edge of the tool, even when the rate of metal-emoval is considerably in excess of that attained when ordinary high-peed steel tools are used. In munitions manufacture, such metallic arbide tool-tips are now in use capable of removing 10 lb. of metal per ninute, the life of the tool between grinds being about 6 hours. Dies vith a hardness approximating to that of the diamond are made from intered carbides, and are extensively used nowadays in the wire and tube rawing industries.

The above remarks refer to examples in which the physical properties f alloys are of special interest, but certain chemical properties, such as esistance to corroding media, sometimes become outstanding desiderata. There are, in general, two ways of securing corrosion-resistance. One is o use, as an alloy-component, passive metals such as platinum, rhodium, nd tantalum. The other method is to select the alloy constituents and heir proportions so that the products of corrosion act in such a way as to orm a protecting barrier between the metal substance and the surrounding nedium. Chromium, for example, is readily oxidized, and the oxide urface-film, being tenacious and continuous, effectively stops progressive xidation. It is such a film, imperceptible to the user, which is utilized n the case of the well-known rust-resisting chromium nickel steel, which ontains 18 per cent. of chromium and 8 per cent. of nickel.

Cast iron is well known to the civil engineer for its uses in dock and arbour work, pipe-lines, etc. During the past 15 years intensive research as explored the whole range of iron-carbon alloys, from these ordinary elatively high-carbon cast irons to the lower carbon members of the range, pproaching the steels. As a result, a series of virtually new cast materials

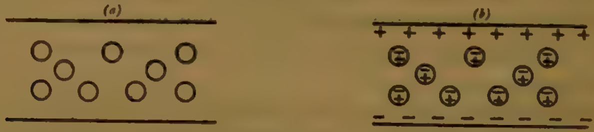
has been evolved, possessing practically any desired ferrous metallurgica structure and having a wide range of properties. Controlled composition with or without alloy additions, heat treatment, and special processing are features in this development, of which the cast crankshaft for auto mobile and other engines forms a characteristic example.

Perhaps I may refer in particular to one of these special cast irons developed by the British Cast Iron Research Association, and consisting of nickel and chromium added to a cast iron containing silicon.[1] A cas has been reported in which a material of this class, Nicrosilal, used in a ta still, has given three times the service of stainless steel without by an means reaching the end of its useful life. Mild steel in the same connexion gave only one-tenth the service of stainless steel.

## The Theory of Dielectric Materials.

I now turn to the electrical section of the range of new materials and select, for more detailed consideration, electrically insulating materials since it is generally recognized that among the various technical problem which confront the electrical engineer, those presented by the limitation of dielectrics are of outstanding importance. Before proceeding, however to mention examples of new materials in this class it may, I think, be helpful if I say a word or two about the theory of dielectric action.

*Figs. 3.*



For this purpose let us consider the case of an electrical condense with a dielectric between its plates, which, for simplicity, we will imagine consists of spherical molecules. In *Fig. 3* (*a*) the condenser is shown un charged, whilst in *Fig. 3* (*b*) charges are shown on the condenser-plates which cause the molecules to become polarized by induction. It will readily be seen that the influence of the polarized molecules, or temporary electric dipoles as we may call them, is such as to neutralize the electric influence of the condenser-plate charges, and the potential-difference between the plates is less than if the dielectric were replaced by a vacuum. Thus a condenser with a dielectric, the molecules of which behave as shown, will hold a greater charge on its plates, for the same potential-difference, than will a condenser without a dielectric. Putting matters in another way we can say that the presence of the dielectric increases the capacity of a condenser.

[1] J. G. Pearce. "Cast Iron Research and the Gas Industry. Trans. Instn. Ga Engrs., vol. 87 (1937–8), p. 986.

Now this simple picture of the dielectric under electric stress as a olarized molecule tells us immediately that its influence in increasing the apacity of the condenser will increase with the number of dipoles per unit olume, and will also increase with the magnitude of the dipole moment. n our search for materials with high dielectric constants, we must therefore e on the look-out for a relatively dense substance and also for one, the olecules or atoms of which polarize readily under the influence of an ectric field.

The influence of density is very effectively shown by a comparison of e values of the dielectric constant of titanium oxide for the three crystal rms in which it is found in nature (Table IV).

TABLE IV.

| | Rutile. | Brookite. | Anatase. |
|---|---|---|---|
| ensity . . . . . . . | 4·21 | 4·11 | 3·87 |
| ean dielectric constant . | 114 | 78 | 48 |

Here we see that the value of the dielectric constant increases with ensity; so that, if we wished to use titanium oxide as a component in an sulator for which a high dielectric constant is required, we should employ e densest form of it, namely, rutile.

The second requirement of the dielectric is that its molecules should olarize readily, that is to say, that the electrons in its atoms should easily e strained by forces slightly out of their usual positions. Under the fluence of alternating electric forces, the electric moment of the dipole anges sign periodically in obedience to the electric field changes. There re, however, a number of substances, the molecules of which incorporate ermanent electric dipoles. Such substances usually possess, as we might xpect, a high dielectric constant. Under the influence of an alternating eld the permanent dipoles tend to rotate, but such motion is subject to species of friction, and energy is dissipated within the body of the dilectric. Solid substances possessing permanent molecular dipoles are erefore not usually suitable for use as dielectrics, because of their high ower-factor. The ordinary electron polarization of atoms under the fluence of electric forces is a frictionless process and so is not accompanied y an energy-loss. Even in the case of substances not containing permanent electric dipoles, however, it is always found that there is some ielectric loss which may be due to the presence of small quantities of oisture between or adsorbed on the surface of internal discontinuities.

Since substances with permanent electric dipoles would be strong echanically because of the high van der Waal's forces between adjacent olecules, a look-out is being kept for substances in which the motion of ch dipoles is restricted under alternating electric forces, and thus internal

loss is prevented. It is interesting to note in this connexion, as pointed out by E. B. Moullin,[1] that in the case of certain organic compounds, the molecules of which embody two equal and oppositely-polarized permanent dipoles, the dielectric loss, as measured by the power-factor, is low.

*Some Practical Examples of New Insulating Materials.*

Before mentioning some examples of new insulating materials it is, I think, important to bear in mind that the properties of any dielectric must be considered in relation to any particular application, since different properties are desirable in different connexions. In high-voltage work, for example, electric strength or resistance to breakdown is all-important; but here it may be noted that such breakdown may be of a surface character. By contrast, the telephone engineer, who employs relatively low-voltage circuits, is not so much concerned with electrical breakdown as with wastage and electrical losses. He requires the insulating material around his telephone cable to have a low dielectric constant, so that the alternating current leakage to earth is as low as possible. He also requires the insulating material to have a low power-factor so that energy is not dissipated as heat in the body of the insulator. On the other hand, the maker of a condenser for high-frequency work seeks materials with a high dielectric constant, so that rapidly-alternating currents will pass through it with little obstruction. Since, however, the rise of temperature in a dielectric (which is, of course, undesirable) is proportional to the product of the dielectric constant, the power-factor, and the frequency, the need for a low power-factor is especially acute when the frequency is high.

It will be remembered that 20 years ago the insulating panels of wireless sets were made of ebonite, a compound which is made by heating rubber and sulphur together. Now whilst ebonite has excellent mechanical and electrical properties, it possesses the grave disadvantage that, on exposure to light and moist air, part of the sulphur becomes oxidized, with the ultimate formation of sulphuric acid. Not only does the surface insulation deteriorate because of this, but also the acid is apt to attack metal terminals and other fittings on the panel.

In modern mass-produced radio sets the insulating parts previously made of ebonite are now made of a filled thermo-setting resin such as bakelite. Although such materials are inferior electrically to ebonite, they have, in addition to their useful moulding properties, the advantage of not suffering surface-deterioration on exposure to light and moisture.

Another panel material which is strong mechanically and will withstand high temperatures is "mycalex", a material consisting of powdered mica bonded with borates of lead and sodium. It is less brittle than glass and can be worked relatively easily mechanically. Although its power-factor

[1] "The Molecular Nature of a Dielectric." Journal Instn. Elect. Engrs., vol. 8 (1940), p. 113 (Feb. 1940).

is not so low as that of mica, it is lower than that of ebonite at radio-frequencies.

For high-frequency work, however, materials with better dielectric properties than those of anything yet mentioned are now available. It is a well-known fact that hydrocarbons are usually good dielectrics, so that it is not perhaps surprising that the thermoplastic substance polystyrene, which has in its constitution no polar groups, was found to have good dielectric properties. It is a glass-like substance with a low dielectric constant and extremely low power-factor. It is therefore an excellent insulating material for the construction of insulating supports in high-frequency work, where the product of the dielectric constant and the power-factor is required to be low. One disadvantage of polystyrene is that it softens at 60° C., solidifying, of course, like other thermoplastics on cooling. It must therefore be drilled rather carefully, water being allowed to drip on the drill to prevent the temperature rising unduly. For use where rather better mechanical properties are necessary, acrylic acid and ketone resins are to be preferred. The electrical properties of these substances are more useful for the construction of bolts, nuts, and screws in apparatus where the use of metal is not permissible or is undesirable. The primary standard of mutual inductance of the National Physical Laboratory is, for example, supported on three levelling screws made from a ketone resin made at the Chemical Research Laboratory. Both the screw and the bushing on which it works are made from the resin, the total load being of the order of 1 cwt.

An interesting example of a dielectric material suitable for use in fixed condensers is the ceramic in which the principal constituent is rutile, the densest crystalline form of titanium oxide. Since rutile possesses a very high dielectric constant (about 170 for electric fields in the direction of the axis and about 90 in a direction at right angles to this), it is possible to make materials, consisting of rutile and a suitable binder, with dielectric constants ranging from 20 to 100.

The rutile ceramics, however, have the disadvantage that the dielectric constant decreases with increase of temperature. Other ceramic materials, such as those made with steatite instead of rutile, show just the opposite effect, the dielectric constant increasing with increase of temperature. It is therefore possible to make a mixture of the two which has a negligible temperature-coefficient.

## The Nature of Ferro-magnetism.

I turn now to the subject of magnetic materials, but before drawing attention to certain practical examples I wish to say a few words about the theory of such materials. Though it is true, as Faraday first showed, that all substances respond to some extent to magnetic forces, the magnetism induced by such forces in metals like iron, cobalt, and nickel is so very

pronounced and exceptional as to require quite separate theoretical con sideration. Why should these elements, which do not exhibit corres ponding abnormalities in their chemical and other physical properties, b so specially privileged as regards their magnetic qualities? After a least a century of theoretical speculation, it is only within the last 1 years or so that the theory of ferro-magnetics, for that is the class-name o these exceptional elements, has shown indications of maturity. Eve now the story is largely qualitative, but I think we can feel fairly confiden about the accuracy of its main features.

Since we know that atoms contain moving electricity in the form o rotating electrons, we have no difficulty in identifying each atom as a elementary "magnet," for a rapidly-moving electron is equivalent to a electric current and must correspondingly be regarded as producing a associated magnetic field. Modern atomic theory, however, tells us tha the motion of the electron is of a two-fold character. Not only does i travel in its orbit round the nucleus, but also it spins about an axis through itself. Both orbital and spin motions are therefore responsible for th magnetic field associated with the atom.

Since atoms behave like elementary magnets, we might expect them t align themselves immediately under the action of an improved magneti field. Owing to the thermal motion of atomic agitation and rotation, how ever, which only disappears at the absolute zero of temperature, such alignment is rendered difficult. Atoms receive blows from all quarter which tend to prevent the maintenance of a common orientation. It i difficult, as we all know, to keep facing one way in a jostling crowd.

In the case of atoms of the ferro-magnetic class, however, it turns ou that there is a spirit of co-operation which is markedly absent in the cas of the rest of the elements. Whole colonies of atoms, even in the absenc of an external magnetic field, are found to align themselves merely as result of interaction between themselves. Such "domains," as they ar called, usually consist of $10^{14}$ to $10^{15}$ atoms occupying about $10^{-8}$ to $10^{-}$ cubic centimetres in volume. In the unmagnetized state these domain are pointing in all directions, so that the external magnetic effect of th solid is nil. When a magnetic field is applied, however, the domain one by one, are turned, until, with increasing field, they are all pointing i the same direction and the specimen is said to be magnetically saturate

In our inquiry concerning the origin of ferro-magnetism we can no put our question again, but this time a little more precisely: why d magnetic domains form more readily in iron, cobalt and steel than in th case of other elements? Since the atomic magnetism arises from th orbital motion or the spin-motion of the electron, it is clear that, in domain, there must be some inter-atomic force which aligns the orbits the spin-axes or both. It was the theoretical physicist Heisenberg[1] wh

[1] W. Heisenberg. "Theory of Ferro-magnetism." *Zeitschrift für Physik*, vol. (1928), p. 619.

1928, first showed that such forces are electrostatic in character and ıat, as a result of them, it is the axes and directions of rotation of the ectron spin which are aligned in a domain. There are no correspond- gly potent electrostatic effects tending to align the electron orbits. We ın thus quite correctly state that we have identified the ultimate and ementary magnetic particle as the spinning electron itself.

These electrostatic forces of alignment, of "exchange interaction" as ıey are termed, are, of course, operative in the case of all atoms; but the sults are different in the case of the ferro-magnetics from those in the case the other elements. The electrons around an atomic nucleus are dis- ibuted in shells which, with increasing atomic number, are filled up from ıe inner shell outwards. Now the aligning forces are particularly strong hen there is a certain value of the quantity: $\left(\dfrac{\text{atomic separation}}{\text{radius of electron shell}}\right)$. or the third shell of the ferro-magnetics, this quantity is of just the quired order and we have the alignment of the spinning electrons in hat orbit.

I am sure that this explanation will recall the magnetic theory of the te Sir James Alfred Ewing,[1] described by him on the first of the two ccasions when he delivered the James Forrest Lecture. If, for his molecular magnets," we now read "magnetic domains," we can retain ıe essentials of his theory, which provides a vivid explanation of such ıagnetic phenomena as hysteresis and saturation.

*ew Magnetic Materials.*

The science of magnetism is sufficiently developed for us to be able to pecify the particular quality we require a magnetic material to possess for s successful use in a particular way. For the core of a telephone trans- ormer, for example, we require the material to acquire an intense mag- etization under a weak imposed field. We require, as we say, a high ermeability. We also wish the magnetization to disappear when the nposed field is removed; that is, the magnetic remanence must be low. oft iron is more suitable than steel in this respect, but the ferro-nickel alloy nown as "permalloy" (containing 78 per cent. of nickel) is better than ither, its initial permeability being 12,000, as opposed to something over 00 for soft iron dynamo-sheet. A further improvement in the properties f permalloy can be produced by the addition of a small percentage of ıolybdenum, chromium, or copper.

For the core of an alternating-current transformer we know that the ıagnetic processes should be as reversible as possible, so that hysteresis ffects do not give rise to the wasteful heating of the core in actual working.

---

1 "Magnetism." Minutes of Proceedings Inst. C.E., vol. cxxxviii (1898–99, 'art IV), p. 289.

Such heating is also restricted if the electrical resistance of the core is high. Magnetic measurements on various materials show that soft iron would satisfy our requirements in the first respect, as its hysteresis-loss per cycle is only of the order of 3,000 ergs per cubic centimetre for a maximum flux-density of 10,000 gauss. For the well-known silicon-iron alloy containing approximately 4 per cent. silicon, due originally to Sir Robert Hadfield, the corresponding figure is only 1,500 ergs per cubic centimetre, whilst the material possesses the additional advantage that its electrical resistance is high. Yensen and Ziegler[1] have shown, however, that by progressively reducing the percentage of impurities—notably carbon and oxygen—the hysteresis-loss in both iron and iron-silicon alloys can be reduced to values of the order of 100 ergs per cubic centimetre. The labour involved in the purification process is very great, however, and such materials are not available commercially. Of commercial materials the nickel-iron alloys again offer the best characteristics, provided that only weak fields are applied, the hysteresis-loss being only about 50 ergs per cubic centimetre for a maximum flux-density of 5,000 gauss, and they are extensively used in current transformers and in transformers for use at high frequencies.

Probably the most striking improvements in magnetic materials, however, are to be noted in those used for making permanent magnets. Here another magnetic characteristic is the measure of suitability. If we magnetize a ferro-magnetic ring specimen to saturation and then cut off the magnetizing current, it is found that the specimen retains a certain amount of its magnetization. We state the magnitude of this retained magnetism in terms of its remanence. If now we reverse the magnetizing force, by reversing the current in the magnetizing coil, we can, by gradually increasing the value of the current, destroy the magnetization in the specimen. In the phraseology of the day, we "degauss" it. The magnetic force necessary to destroy the magnetization is called the "coercive force." Evidently a material with a high coercive force will tend to maintain its magnetism in spite of adverse magnetic influence. Now a permanent magnet tends to "degauss" itself, because of the action of its own free north and south poles. That is why, in the past permanent magnets were made as long as possible. The discovery of new materials with high coercive force now enables us to make much shorter and more permanent magnets. Whereas at one time carbon steel was the only available substance of this type, we now have a number of new alloys which are greatly superior in performance. First of all it was discovered that tungsten steel had a higher coercive force than any previously known substances. Then it was found that a 35-per-cent. cobalt steel was still better. Such cobalt steel is now much used for the per

[1] T. D. Yensen and N. A. Ziegler. "Magnetic Properties of Iron as affected by Carbon, Oxygen, and Grain-Size." Trans. Amer. Soc. Metals, vol. 23 (1935), p. 55 (June 1935); "Effect of Carbon, Oxygen, and Grain-Size on Magnetic Properties of Iron-Silicon Alloys." *Ibid.* vol. 24 (1936), p. 337 (June, 1936).

manent magnets of wireless sets. Eight years ago it was discovered in Japan that an alloy of approximately two parts of iron to one of nickel and one of aluminium, cooled from the melt in a certain way, was superior to anything yet discovered. A small magnet made of this alloy is capable of supporting a weight of 56 lb., although its own weight is about 2 oz.

Still further improvement has been effected by the addition to the iron-nickel-aluminium alloys of small percentages of cobalt and copper. The resulting complex alloy, known as "alnico", provides the most powerful permanent magnet material at present available commercially, although claims have been made in Japan that an alloy of iron, cobalt, nickel, and titanium has an even larger coercive force.

It cannot yet be said that the theoretical physicist is ready with a complete explanation of the origin of the remarkable properties of these new magnetic materials. Using the weapon of X-ray analysis, however, he has been able to distinguish clearly between the different solid structures which result from cooling liquid alloys at different rates, and has already been able to recognize that a high state of internal local strain within the body of the material appears to be necessarily associated with the exceptional coercive force. The new permanent-magnet alloys were, it is true, discovered by empirical methods, but further systematic study of the relation between internal properties and molecular behaviour may well be expected to lead to the synthesis of combinations of elements with even more remarkable characteristics.

## Conclusion.

I have now completed what, I fear, has been but an inadequate survey of the way in which scientific inquiry and experimental measurement, with now and again an empirical enterprise, have led to the discovery of better materials for practical use. Physical science, as we know it to-day, grew originally out of the study of practical lore, and even since it began to stand on its own feet as an independent theoretical interpretation and shorthand description of nature, it has often received fresh stimulus by coming back to practical problems. The study of the solid state is now proving a fertile, if yet a difficult, field of inquiry for the theoretical worker, and there can be little doubt that when we possess a deeper understanding of the ways in which atoms and molecules cohere to form solid substances, both simple and compound, we can expect such fundamental knowledge to point the way to yet further practical improvement. The philosopher Francis Bacon[1], about 300 years ago, used to distinguish between those results of science which proceed from light-giving experiments (*experimenta lucifera*) and those which follow from experiments of practical utility (*experimenta fructifera*). It is true that the theoretical worker is consumed

[1] *Novum Organum*, Book I, Aphorism xcix.

mainly by his curiosity to know and understand all about things. He desires to bask, shall we say, in the light of understanding. But the results which Bacon called *lucifera* certainly provide the surest, and often indeed the quickest, way to the other kind of results which he called *fructifera*. Yet, in the partnership of science and practice, the assistance rendered is by no means always in one direction. Science has grown out of practical knowledge and it has nothing to gain, but indeed much to lose, by forgetting or neglecting to seek further inspiration and assistance from its origins.

In conclusion I wish to record the friendly counsel and assistance received from many of my colleagues of the Department of Scientific and Industrial Research in the selection of the subject matter of this discourse.

**Sir Leonard Pearce**, in moving a vote of thanks to the Lecturer, observed that Dr. Appleton's address discussed the very fundamentals of the profession, since theory and practice depended to a high degree upon the solid materials available for design and construction; the discourse had dealt with materials from the Steel Age through that of plastics to that of alloys. In connexion with the subject of plastics and synthetic materials, to which a large proportion of the Lecture had been devoted, it was of great interest to hear of the remarkable physical properties and physical strengths of certain plastics, and it was perhaps opportune to refer to the impetus given to the subject of materials and their testing by the setting up of a Joint Committee in Great Britain, in whose work no less than 26 per cent. of the electrical Institutions and societies were associated. The subject of plastics had been selected for the next symposium. The Lecturer had also dealt with the use of dielectrics in electrical engineering, which had exercised a profound influence upon modern high-tension cable systems and transmissions. The Lecturer had dealt with alloys with special regard to their magnetic properties. It was perhaps too much to hope that, having regard to the extensive ground covered by the Lecturer, he would find himself able to deal subsequently with the question of steel alloys, which, by their properties of resistance to creep and oxidation, had played such a profound part in the development of the steam cycle, resulting in striking increases of pressure and temperature, with consequent enormous savings in the fuel-consumption required for power-production.

**Mr. Asa Binns**, in seconding the motion, gave instances from his own experience, showing how difficult it was for a practising engineer to keep up to date with the rapid advances in fundamental science. Dr. Appleton had performed a valuable service to the profession in bringing to light subjects of which engineers were aware, but which they were inclined to neglect.

The motion was carried by acclamation.

## ORDINARY MEETING.

11 June, 1940.

Sir CLEMENT DANIEL MAGGS HINDLEY, K.C.I.E., M.A., President, in the Chair.

The Council reported that they had recently transferred to the class of

*Members.*

MURRAY BARCLAY BUXTON, M.C., M.A. (*Cantab.*).
PHILIP LOCKTON FORWOOD, M.A. (*Cantab.*).
ROBERT LEWIS MCILMOYLE.
CHARLES ARTHUR RISBRIDGER, B.Sc. (Eng.) (*Lond.*).
WALTER SEPTIMUS STREDWICK.

and had admitted as

*Students.*

ARTHUR IAN ADAMS.
MICHAEL OLIVER BARRETT.
ERNEST HUGH BEARE.
DAVID WALTER BISACRE.
ILLIPARAMPIL GEORGE CHACKO.
JOHN CHATTAWAY.
RONALD WILLIAM CROKER.
MACDONALD STUART GEORGE CULLIMORE.
JOHN DAVID.
ALAN LAWRENCE EAGLES.
ARTHUR GEOFFREY EDWARDS.
RONALD BERNARD ELLIOTT.
RICHARD JOHN GODDEN.
BARKAT RAM GOEL.
ROBERT MORPHET HAYTHORNTHWAITE.
JOHN EDWARD VICTOR HOLMES.
JACK HUGGINS.
DERRICK GRAHAM HURTLEY.
HENDRIK JOHANNES JOUBERT.
ROBERT DOUGLAS KIRKPATRICK, B.Sc. (*Witwatersrand*).
PETER LAWSON.
JOHN GWYNNE LLOYD, B.Sc. (Eng.) (*Lond.*).
JOHN ALLISTER LOE.
JAMES PRYDE MCBEATH.
WILLIAM ERIC MCTRUSTY.
PAUL CECIL MARKS.
LEONARD MARMION.
ELIJAH MIRKIN.
FRANCIS VICTOR MONTGOMERY, B.Sc. (Eng.) (*Lond.*).
JOHN HOWARD MOON.
ARTHUR PHILIP NEWELL.
GEORGE HERBERT NICHOLLS.
HUGH OGILVIE PATERSON, B.Sc. (*S. Africa*).
EDWARD PHILIP PEARN.
KEITH NEWTON POWELL.
ROBERT EDWARD ROGERS.
JOHN GERARD SHERRY.
BRYAN JOHN SMITHSON.
PERCY GORDON SPENCER.
WILFRID ALAN STOKES.
ANDRÉ TCHERNAVIN.
VETTIVELU THAMBIPILLAY.
KANDIAH THURAISINGHAM.
NOEL VINCENT BLUNDELL WHITEHOUSE.

**The President** put to the Meeting a recommendation by the Council that Sir Edwin Landseer Lutyens, K.C.I.E., President of the Royal Academy since 1938, be elected an Honorary Member of The Institution. He observed that Sir Edwin Lutyens was probably the most distinguished architect of the time. The Institution was especially indebted to him for the advice that he had given in the laying-out and planning of the Benevolent Fund Homes at Haywards Heath ; he had taken great interest in that work, which he had carried out in an entirely honorary capacity.

The recommendation was agreed to by acclamation.

The Scrutineers reported that the following had been duly elected as

*Associate Members.*

GEORGE ROUGH ADAMS, Jun., B.Sc. (*St. Andrew's*), Stud. Inst. C.E.
WILLIAM ALLEN, B.Sc. (*Belfast*).
JOSEPH EVELYN FERNON ANDERSON, B.Sc. (*Manchester*).
JOHN APSE.

Ewart Kenneth Astin.
Ronald Ernest Dudley Bain, B.A., B.A.I. (*Dubl.*).
Jai Dev Batra.
Reginald Walter Bishop, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
John Scobell Boissier, B.Sc. (*Birmingham*).
John Bolton, M.Eng. (*Liverpool*).
Thomas George William Boxall, B.Sc. (Eng.) (*Lond.*).
Herbert James William Braddick, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Jack Edward Butt.
Laurence Humphrey Carder.
James Andrew Cashin.
William Edwin Charles Chamberlain.
Donald Duncan Joseph Clarke, B.Sc. (Eng.) (*Lond.*).
Charles Leslie Clayton, Stud. Inst. C.E.
Leonard Evan Losack Coleman, B.Sc. (Eng.) (*Lond.*).
Harry Collins.
Donald William Cracknell, Stud. Inst. C.E.
Denys Walton Crawshaw, Stud. Inst. C.E.
Norman Callow Cregeen, Stud. Inst. C.E.
David Simpson Cuthill, Stud. Inst. C.E.
William Davies, B.Sc. (*Glas.*).
Albert de Barr.
Nicholas Charles Callard de Jong, B.Sc. (Eng.) (*Lond.*).
Sidney Favel, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Christopher Evelyn Fenwick, M.Sc., B.E. (*New Zealand*).
Edward Fish, B.Eng. (*Sheffield*).
Ernest Reginald Gambrill.
Charles Black Glenesk.
Leslie Gordon, B.Sc. (*Edin.*), Stud. Inst. C.E.
Arnold Marshall Greenwood.
John Eric Guest, Stud. Inst. C.E.
Alfred Hammerton.
John Henry Roderick Haswell, B.Sc. (Eng.) (*Lond.*).
John Marshall Hitchen.
Albert Leslie Hobson, B.Sc. (Eng.) (*Lond.*).
George Ernest Hollings.
Leslie Ernest Hunter, M.Sc. (Eng.) (*Lond.*).
Percy William Hyde, B.Sc. (*Leeds*).
George Harold Banks Jacques.
Bernard Walter James, B.Sc. (*Edin.*).
Allan Lister, B.Sc. (*Leeds*).
Gilbert Little, B.Sc. (*Glas.*).
Lloyd Hepworth Mansfield.
Harry Herbert Margary, M.A. (*Cantab.*), B.A. (Commerce) (*Manchester*), Stud. Inst. C.E.
Denis Dearman Matthews, M.A. (*Oxon.*), M.Sc. (Eng.) (*Lond.*).
John Hutton Matthews.
David Croll Milne, B.Sc. (*Glas.*).
Thomas Lyle Morgan, B.Sc. (*Wales*).
Alfred Edward Murray, B.Sc. (*Aberdeen*).
Ronald Needham, B.Sc. Tech. (*Manchester*).
James Thomas Noble, Stud. Inst. C.E.
Robert Evers Normanton.
James Osenton.
John Theodore Partington, Stud. Inst. C.E.
Frederick Noel Brewster Patterson, B.Sc. (*Durham*), Stud. Inst. C.E.
Richard Pavry, B.Sc. (Eng.) (*Lond.*).
William Glanvill Phillips, B.A. (*Cantab.*), Stud. Inst. C.E.
John Albert Posford, B.A. (*Cantab.*), Stud. Inst. C.E.
Frank Vernon Powell.
Kanuru Lakshman Rao, M.Sc. (*Madras*).
Eric George Robins.
Norman Sibley Robinson.
William Rolinson.
George Alexander Rotinoff, M.A. (*Cantab.*).
Cumaraswamy Sabaratnam, B.Sc. (*Glas.*).
Ernest Harold Sidwell, Stud. Inst. C.E.
Leonard James Simpson.
Roy George Sydney Smart.
William Russell Smellie, B.Sc. (*Glas.*).
Harry Clifford Smith.
Herbert Eaton Stone, Stud. Inst. C.E.
Harold Charles Swaffield, B.Sc. (Eng.) (*Lond.*).
Charles Twynam Teychenné, M.C.
Wilfred Arscott Tilbrook, M.Eng. (*Sheffield*), Stud. Inst. C.E.
Joseph Turnbull.
John Brian Walton, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
James Sanderson Wardell.
Harold George John Watson.
Sidney George Keevil Wright, Stud. Inst. C.E.

# ANNUAL GENERAL MEETING.

11 June, 1940.

SIR CLEMENT DANIEL MAGGS HINDLEY, K.C.I.E., M.A.,
President, in the Chair.

**The President,** in moving that the Report of the Council for 1939–40, as published in the June 1940 number of the Institution Journal[1], be taken as read, observed that it had been usual to read the Annual Report at the Annual General Meeting, but the Council had thought it advisable under present circumstances to dispense with that reading and to publish the Report in advance in the Journal.

**Mr. R. G. Hetherington** seconded the motion.

The motion was agreed to by the members present.

**The President,** presenting the Report of the Council for 1939–40, said that it was laid down in the By-laws that at the Annual General Meeting the Report of the Council should be received and deliberated upon. He wished to take the opportunity to comment on a few of the more important points dealt with in the Report, and then he would propose that the Report be received and approved. After that resolution had been seconded, it would be open to any member to raise any questions on and to discuss the Report, and then the resolution would be put to the meeting.

The procedure that was being adopted at the meeting was, in fact, a reversion to the procedure which The Institution had followed in its early days, when it was customary for the President to review the work of the session in an Address to the members at the close of the session. In course of time that review became the Annual Report, and in 1839 the custom whereby the President addressed The Institution at the beginning of the session came into force. It was not until 1856 that it became usual for the President to address The Institution on some specific aspect of engineering. Unfortunately, that custom was one of those which The Institution had had to forego owing to the outbreak of war. In spite of the fact that it had been necessary to discontinue meetings during the early part of the war, it was found possible to resume meetings subsequently and to carry out a satisfactory programme during the session, which had been extended to the end of May.

The President reminded the members of the award of the James Alfred Ewing Gold Medal for Engineering Research to Professor G. I.

[1] Journal Inst. C.E., vol. 14 (1939–40), p. 345.

Taylor, and of the award of the Charles Hawksley Prize to Mr. W. E. Blackmore, a Student, for his design of a water-tower in reinforced concrete. That design was an excellent one, and it was an encouraging sign that the Prize should have been won by a Student in the face of somewhat keen competition.

The war had rendered it necessary to cancel the Annual Dinner and the 1940 Conversazione, but The Institution was able to arrange a Luncheon on the 19th April 1940, and was greatly honoured by the presence of Sir John Anderson, Home Secretary, and Sir John Reith, then Minister of Information. Those gentlemen had addressed the members of The Institution at the Luncheon, and their speeches had been reported in the Institution Journal for June 1940. It was worth recalling the very high tribute which Sir John Anderson had paid to Civil Engineers for their assistance in civil defence work generally, and he (the President) would like to remind members of the stimulating and encouraging words in which Sir John had directed attention to the part which engineers would have to take in post-war reconstruction.

The cancellation of the American visit had been a matter of great disappointment; but there had been a very happy sequel in the informal Luncheon at which the Councils of the Institutions of Civil and Mechanical Engineers had entertained Mr. Kennedy, the American Ambassador, when he had presented the Diploma of Honorary Membership of the American Society of Civil Engineers to Mr. W. J. E. Binnie, the late President of The Institution, whilst a similar honour had been conferred on the same occasion by the American Society of Mechanical Engineers on Mr. Bruce Ball, the late President of the Institution of Mechanical Engineers. That was a welcome sign of the cordial relations which existed between the Institutions of Civil and Mechanical Engineers, and also between those Institutions and the great engineering societies of America. The occasion was the first upon which the Councils of the two British Institutions had met together in the same room, and those present had so much enjoyed the experience that it was hoped that it would be repeated.

Before leaving the outward and visible signs of the vigorous life of The Institution, the President paid tribute to the Local Associations in Great Britain, which, with one exception, had been able to hold meetings, and to the Overseas Associations, which had been carrying on quite successful programmes and had shown very marked interest in co-operation with the local branches of other engineering Institutions. It was the Council's desire to encourage that co-operation in every way possible, and it was particularly of value for the Overseas Associations to cultivate close relationships with the national engineering bodies established in the countries in which those branches were located.

Coming to matters which were more concerned with the internal administration of The Institution than with its outward manifestations, the President said that the war had introduced many problems for the

Council and the administrative staff of The Institution, whose numbers were considerably depleted and who had had imposed upon them a good deal of heavy work. In the continually changing conditions the Council had determined to lose no opportunity of watching over the education and training of the young men and to do everything possible to influence those in power to ensure that those young men should be able to complete their preparation for the profession and that they should be employed in spheres of action best suited to their attainments. There was in the Annual Report a brief notice with regard to that work; but the President assured the members that that represented only a fraction of the work which had been handled by the Secretary and his staff in pursuance of the object of the Council. The main aim had been to ensure, as far as possible, that there was no interference with the Students' education and training until they had to be called up for national service; the Council felt that it was worth every effort to secure the continuity of the profession by safeguarding as far as possible the Students and the young members so that they should receive their full training. The Institution did not claim any special privilege for its members and Students, except the privilege of being employed in the national interest in places where their knowledge and experience would be of most use. The Institution recognized the assistance that had been given by the Government, the Military authorities, and the various educational bodies towards this end. With regard to placing engineers in posts of national importance, a great deal of work had been done by the Ministry of Labour in administering the Central Register. Mr. S. B. Donkin, Past-President Inst. C.E., was in charge of the Committee of the Engineering Section of the Register and the burden of work which devolved upon him and upon Mr. Clark, the Secretary of The Institution, in dealing with the references made to The Institution, was by no means light.

The Institution had tried to maintain so far as possible records of its members who were serving with H.M. Armed Forces. That work depended very largely upon information supplied by the members themselves, so that it could not be expected that the records were quite complete. So far, however, the records showed that, of the 1,018 members of whom The Institution had information, 30 Members, 363 Associate Members, 2 Associates, and 623 Students were serving, of whom 357 held commissions in the Corps of Royal Engineers, and 158 were training for Royal Engineer commissions, whilst there were 161 other ranks in the Royal Engineers. In other branches of the Services, 151 of those on the Roll of The Institution held commissions and 113 held non-commissioned rank. Of those who were serving, 66 per cent. were with the Royal Engineers.

Dealing with certain changes that had been made in the By-laws, details of which had been published in the Institution Journal, the President said that those changes had been made especially to meet emergency conditions during the war, in order to render it possible to submit for election

the names of people who otherwise, under a strict interpretation of the By-laws as they had existed at the outbreak of war, would have been debarred. In that way the Council had obtained considerable power to make exceptions and exemptions. It was, however, their fundamental policy that the By-laws relating to admission of members should be administered with strict regard to the preservation of the standards of The Institution. It was not intended in any way that persons should be presented for election unless the Council were satisfied that they had been adequately educated and trained.

It had been suggested in some quarters that a body such as The Institution of Civil Engineers, large and powerful as it had become, might well make an even greater and more direct contribution to the war effort than had hitherto been found possible. It was true that in normal times The Institution could act in certain directions with the full force of its large membership, and could make use of the accumulated knowledge and experience of that membership. In war-time the very large majority of the members were individually engaged in important duties and were contributing to the full their individual effort. Consequently, The Institution could not mobilize any large number for special work, as could be done in the great industries which had powerful organizations of employers and of labour. The Institution had no organization of the kind that would enable it to allot its members to specific work for the assistance of the Government, although it could assist in any specific problem by placing before the Government the names of members who had special knowledge and experience. The President emphasized, however, that The Institution was ready to do anything that it might be asked to do, and that it would lose no opportunity of carrying out any duty which the Government might require of it. In pursuance of that policy, it had recently offered its services, jointly with the Institutions of Mechanical and Electrical Engineers, to the Prime Minister for any work of which the Institutions or their members might be capable. A reply had been received from the Prime Minister, from which the Institutions knew that the offer had been very gratefully received and was being considered by the Departments principally concerned with the war effort. It was well to remember that the contribution made by The Institution to the war effort was the sum of the individual efforts made by all its members, in whatever capacity they were called upon to work, and they could all feel great satisfaction and pride in the fact that the care with which The Institution had watched over their education and training, and their selection as members of The Institution, had had a profound influence upon the class of service they could give.

The Accounts had been published with the Annual Report[1], and he wished to draw attention to the financial strength of The Institution. The total expenditure involved in purchasing the site and erecting an

[1] Journal Inst. C.E., vol. 14 (1939–40), p. 362. (June, 1940).

completing The Institution's magnificent building (namely, £375,767), had been completely met out of The Institution's resources. The year covered by the Accounts was the first in which there was no entry for the repayment of the capital loan obtained for the Building Fund, the final payment having been made in 1939. In addition to the building, which The Institution possessed freehold, and its contents, there were some £50,000 of investments and a cash balance of more than £12,000 towards the expenditure in 1940. There was also, in various Trust Funds, about £38,000 invested, and £3,300 of unexpended income. He had no means of estimating the value of the Institution building, of the site, the valuable furniture and fittings, the Library of more than 60,000 volumes, and the many works of art and historical relics of great intrinsic value; but he did not think that it would be an exaggeration to place the value in normal times at considerably over half a million pounds sterling. The Institution was therefore a body owning a freehold and entirely unencumbered property worth perhaps half a million pounds sterling, having invested funds, whether free or in trust, of £88,000, and an income of about £45,000 annually, derived from the 13,000 members and students on the Roll.

If he were the Chairman of a Company owned by the shareholders for their own benefit, that would not be a bad situation to describe to them. How much greater should be the satisfaction of the members of The Institution when they remembered that their great enterprise, with its material assets and its financial strength, was dedicated to the advancement of science, and that it had been built up by the unaided efforts of the members, from the days when they were few in number and of limited personal means, to the present time, when the number on the Roll was 13,125, of whom more than 10,000 were corporate members and fully-qualified engineers. It was not out of place to mention that for the first time The Institution had reached the 10,000-mark in respect of corporate members, and that it was the first of the great Institutions to reach that figure. It was also of interest to record, when talking of the great achievements of The Institution in the 122 years since its inception, that those achievements were the result of the work of the 22,500 persons who had been elected to membership since The Institution was formed. He felt confident that, with the results of the work of the 12,500 former members, to whom The Institution owed a great debt of gratitude, and the continued efforts of the 10,000 present members, backed by the material resources he had mentioned, The Institution could look forward to a great future when the present difficult days had passed.

In conclusion the President said:—

"It is not in any spirit of boastful pride that I have given you this picture of the Institution's prosperity and material resources because, as I have often told you before, the prestige, the influence, and the very future of The Institution itself depend entirely on the strength and determination which you, its members, put into the work of maintaining,

in all circumstances, the high technical and ethical standards which we have had entrusted to us by our predecessors. In so far as you who are here to-day can influence the future policy of The Institution I ask you to put in the forefront of everything that is done the maintenance of these standards and the traditions which we have been taught.

"At this critical moment in our country's history, in the midst of events which might well lead some to despair of the future, there is this at least on which we can rely. Whatever may happen, whatever material destruction may occur, the standards which our Institution has established are of an abiding nature, not dependent upon material conditions. They are a possession which no one and no circumstances can take from us, so long as we put them in the forefront of all our work."

**The President** then formally moved that the Report of the Council be received and approved.

**Mr. R. G. Hetherington** seconded the motion.

**The President** invited questions and discussion on the Report.

**Mr. P. J. H. Unna** said that he hoped that the practice of circulating the Annual Report and the Accounts before the Annual General Meeting would be continued.

Unfortunately, the investments of The Institution included considerable railway securities; it might be advisable to cut some of the losses and to spread the risk, although the present time might not be suitable. The Trust Fund investments amounted to more than £35,000. Could not a consolidated fund be formed which could be invested in another type of security, possibly with increased income?

More pages might be kept out of the bindable part of the Institution Journal, and put into the unbindable part. For example, of the 140 bindable pages of the Journal for June 1940, twenty-eight covered the Annual Report and Accounts, which he believed were to a large extent of temporary interest only. Similarly, two pages were devoted to a list of admissions, which list became redundant when the next list of members was published.

**The President** said that the Council would take note of those suggestions and would deal with them carefully. The present moment was not, of course, opportune for dealing with investments.

**Mr. Robert Chalmers** referred to a case wherein the By-laws or their interpretation appeared to bear hardly.

**The President** pointed out that the By-laws made provision for such a case.

**The President** then moved that the Annual Report be received and approved.

**Mr. R. G. Hetherington** seconded the motion.

The motion was agreed to by the members present.

The Scrutineers reported the election of the Council for 1940–41 as follows [1]:—

*President.*

*Sir* LEOPOLD HALLIDAY SAVILE, K.C.B.

*Vice-Presidents.*

Professor Charles Edward Inglis, O.B.E., M.A., LL.D., F.R.S.
Sir John Edward Thornycroft, K.B.E.
David Anderson, LL.D., B.Sc.
Francis Ernest Wentworth-Sheilds, O.B.E.

*Other Members of Council.*

Sir Athol Lancelot Anderson, K.C.B.
Asa Binns.
Walter Miller Campbell (*South Africa*).
Raymond Carpmael, O.B.E.
Sir Harold Nugent Colam, B.A. (*India*).
Professor Gilbert Cook, D.Sc., F.R.S.
Sir Harley Hugh Dalrymple-Hay.
Jonathan Roberts Davidson, C.M.G., M.Sc.
Charles George Du Cane, O.B.E., B.A.
Richard John Durley, M.B.E., B.Sc., Ma.E. (*Canada*).
Thomas Peirson Frank.
Ralph Freeman.
William Henry Glanville, D.Sc., Ph.D.
William Thomson Halcrow.
Roger Gaskell Hetherington, C.B., O.B.E., M.A.
Ralph Frederick Hindmarsh.
Drummond Holderness (*New Zealand*).
Cecil Lee Howard Humphreys, T.D.
Robert John Mathison Inglis.
Gerald Lacey, B.Sc. (*India*).
William Henry Morgan, D.S.O.
Sir Standen Leonard Pearce, C.B.E., D.Sc.
Joseph Newell Reeson (*Australia*).
Vernon Alec Murray Robertson, M.C.
Alec George Vaughan-Lee.
Herbert Cecil Whitehead.

**Mr. P. J. Cowan** proposed—That the thanks of the Meeting be given to the Scrutineers, and that the ballot-papers be destroyed.

**Mr. Frank Gill** seconded the motion, which was carried unanimously.

**Mr. J. S. Wilson**, responding on behalf of his fellow scrutineers and himself, expressed appreciation of the vote of thanks. He observed that, of 7,406 ballot-papers issued, only 1,493 were returned, and some of those that were returned were invalid. The increase of postage rates had no doubt had an effect on the returns, and the envelope-shortage had led

[1] The Council commence their term of office on the first Tuesday in November, 1940.

to another occurrence. Those members who, apparently, had not regarded the ballot sufficiently seriously had taken the opportunity to enclose, with their ballot-papers, correspondence to the Secretary; when they realized that their envelopes had remained unopened for a couple of months or so, they could not blame the Secretary for not having replied to their correspondence!

**Mr. W. T. Halcrow** moved that the thanks of The Institution be accorded to Mr. E. W. Monkhouse, M.V.O., M.A., M. Inst. C.E., and that he be re-appointed Honorary Auditor for the current financial year; and that Sir Alan Rae Smith, O.B.E., be re-appointed professional Auditor.

**Sir Athol Anderson** seconded the motion, which was carried unanimously.

**Sir Alan Rae Smith,** expressing his thanks for his re-appointment as professional Auditor to The Institution, said that it was a very great honour and privilege to him to occupy that post.

The List of Awards made by the Council for session 1939–40 was announced as follows :—

For Papers read and discussed at meetings :

Telford Premiums to :—

J. E. Bostock, O.B.E., M. Inst. C.E., for his Paper "Remodelling of the Assiut Barrage, Egypt."

William Barnes, for his Paper on "The Dragline Excavator."

Jack Duvivier, B.Sc.(Eng.), M. Inst. C.E., for his Paper on "Cliff-Stabilization Works in London Clay."

The Coopers Hill War Memorial Prize to :—

J. A. R. Bromage, M. Inst. C.E., for his Paper on "The Sewage-Disposal of Delhi."

A Manby Premium to :—

F. A. Rayfield, Assoc. M. Inst. C.E., for his Paper on "The Engineer's Part in the Promotion of Road Safety."

For Papers published in the Journal without oral discussion :

Telford Premiums to :—

A. M. Hamilton, B.E., M. Inst. C.E., and E. B. Cocks, B.E., Assoc. M. Inst. C.E., jointly, for their Paper on "Some Aspects of Aero-Hangar Design."

Robert Walton and T. D. Key, M. Inst. C.E., jointly, for their Paper on "Application of Experimental Methods to the Design of Clarifiers for Waterworks."

J. R. Daymond, M.Sc., Assoc. M. Inst. C.E., for his Paper on "The Hydraulic Problem Concerning the Design of Sewage-Storage Tanks and Sea-Outfalls."

Herbert Chatley, D.Sc., M. Inst. C.E., for his Paper on "The Principles of Drag-Suction Dredging."

R. J. Cornish, M.Sc., Assoc. M. Inst. C.E., for his Paper on "The Analysis of Flow in Networks of Pipes."

Lieutenant-Colonel Rawdon Briggs, D.S.O., M.C., R.E., for his Paper on "The Haifa–Baghdad Road."

H. C. E. Cherry, M.Sc., M. Inst. C.E., for his Paper on "The Subterranean Sources of Water in the City of Rangoon."

<u>For a Student's Paper read at a Meeting of a Local Association:</u>

The James Forrest Medal to :—

D. F. Wilkin, B.Sc., Stud. Inst. C.E., for his Paper on "Concrete and the Resident Engineer." (Northern Ireland.)

A Charles Hawksley Prize of £150 to :—

W. E. Blackmore, Stud. Inst. C.E., for his design of a water-tower.

Having regard to the good work shown by A. R. Collins, M.Sc., Assoc. M. Inst. C.E., and J. K. McIntyre, Stud. Inst. C.E., they have received honourable mention and have been granted £30 and £20 respectively.

A Bayliss Prize to :—

W. E. Blackmore, Stud. Inst. C.E., awarded on the result of the October 1939 Examination.

**Sir George Humphreys**, Past-President, proposed—That the thanks of this Meeting be accorded to Sir Clement Hindley, President, for his conduct of the business as Chairman of the Meeting. The members, he said, had had happy experiences of his Presidency, and they were grateful to him also for his Chairmanship of the Annual General Meeting. He had given them a most interesting and inspiring address, and they wished to thank him very much.

**Mr. N. G. Gedye** seconded the resolution, which was carried with acclamation.

**The President** briefly responded to the resolution.

The proceedings then closed.

# The Resistance to Collapse of Structures Under Air Attack.[1]

By Professor JOHN FLEETWOOD BAKER, M.A., Sc.D., D.Sc. Assoc. M. Inst. C.E.

## INTRODUCTION.

THE necessity of ensuring that the buildings in factories are as resistant as possible to collapse, when subjected to air attack, should need no stressing. Whilst a direct hit on a factory can scarcely fail, by destroying plant or stores, to interfere to some extent with production, the damage will be much more widespread and costly if, at the same time, a main member is weakened and leads, as is possible, to the collapse of a large area of the structure.

Thanks to the somewhat conservative values of working stresses hitherto assumed in design in Great Britain, the great majority of modern buildings, although designed without any thought of attack from the air, will be comparatively resistant to collapse even when subjected to serious structural damage. Certain types could, however, be made much more resistant by slight amendments in design and detail which would not increase the weight of the structure appreciably.

No attempt is made in this Note to deal with particular problems, but the attention of the designer is drawn to general points which he should consider when designing new structures, or when strengthening existing buildings.

The structures discussed fall into four main classes :—

(*a*) Fully-framed steel or reinforced-concrete multi-storey buildings.
(*b*) Single-storey modern steel factory buildings.
(*c*) Older buildings, often partly framed and partly wall-bearing.
(*d*) Special types, such as erection-sheds, with very long spans.

## FULLY-FRAMED MULTI-STOREY BUILDINGS.

Fully-framed steel or reinforced-concrete buildings are comparatively resistant to collapse, since they are capable of adjusting themselves to heavy overloads for which they were not specifically designed.

A near hit is unlikely to damage the main frame seriously. It may demolish panel walls, and these may fall inwards and produce a debris load, which may cause collapse of an already heavily-loaded floor. Such a

[1] Reprinted by permission of the Research and Experiments Branch of the Ministry of Home Security.

collapse will not be produced if, as is often the case, the actual floor load carried is considerably less than that for which the floor was designed. For instance, it has been possible to suggest, when dealing with basement shelters, that no strengthening of a framed building, of steel or concrete, is needed :—

(i) where the actual load carried does not exceed 25 per cent. of the superimposed load for which the floor was designed ;

(ii) if the floor was designed for a load of at least 80 lb. per square foot.

In a factory the actual load may often exceed 25 per cent. of the design load, but that design load may considerably exceed 80 lb. per square foot. When this is so, the percentage can be increased above 25 per cent. As a rough guide, it may be laid down that if a heavy machine rests on a floor-panel and imposes on that panel a load approximately equal to the design load, the structure below that floor-panel should be strengthened. The structure can be most economically strengthened, where space is available, by propping main beams and floor-slabs. In the case of reinforced-concrete frames some care must be exercised in this propping, and a knowledge of the position of the reinforcement is necessary ; but the danger from high shear stresses at the prop is not so acute as is usually supposed. Tests are being carried out on the strength of propped reinforced-concrete beams, and more definite recommendations will be available in the near future.

Whilst a direct hit may wreck one bay of one floor, or more, complete collapse of a large part of the building is unlikely to follow even if some main members are cut. Debris is liable to fall in other bays, however, and in order to reduce the structural damage, strutting as described above should be considered in inside bays, even though no panel walls can fall on them.

The resistance to collapse of framed buildings is due to the continuity in these structures, which is of a high order even in a steel frame with the usual comparatively light cleated connexions. It may be said that wherever continuity can be introduced, it is desirable. For instance, in many factories there are gallery structures carrying machines, the beams being supported on steel stanchions. Usually the beams are not continuous over the stanchions, but are connected into them or joined over them, as a rule by web-plates. If these connexions were made to develop the whole strength of the beam, by welding or otherwise, resistance to collapse would be increased considerably, since in many cases where this provision is made, complete failure of the beam would not follow even if a stanchion were cut.

### Single-Storey Modern Steel Factory Buildings.

This very common type consists of lines of stanchions carrying roof girders—joist beams in small shops and heavy trusses in larger structures—upon which the roof trusses rest.

This type is not particularly vulnerable, but if an internal stanchion be cut the girders on either side will collapse, and may often involve an area of roof 100 feet by 50 feet, or more. In many cases a hit from a large splinter would cripple a roof girder and might bring down the roof trusses supported on it.

These structures could be made much more resistant to collapse. In new construction this would involve little or no additional expense. In existing shops the alterations needed would depend upon the details of the structure, but, in most cases, it should be possible to do a great deal at no great expense. Such work may necessitate the use of unorthodox joints.

It must be remembered that these structures are designed to support the dead load of the roof, with wind loads and snow loads in addition, without a certain permissible stress, far below the yield-stress of the material, being exceeded. What should be required of them, in addition, is the capacity of supporting the dead load of the roof only when any one stanchion or any one main roof girder is cut by a direct hit. In this connection the usual permissible stresses can be safely exceeded. The simplified methods of calculation, used generally in design, are not well suited for the determination of the capacity of the structure to stand when damaged. An examination of the real strength of the structure must be made. It should be remembered, for instance, that failure does not necessarily take place when the yield-stress of the material is developed at the section. It is probable that in many cases no increase in the weight of material in the main structure will be needed to ensure that collapse does not take place when one member is cut. Details, such as the connexions from beam to stanchion, will, however, need revision, and the most economical method will probably be to make the beams or trusses continuous over the stanchions.

It has been stressed above that continuity is very desirable in structures which may have to resist air bombardment. The most efficient type is the rigidly-jointed frame or portal. In steelwork the joints can be made by riveting, bolting, or welding. Attention should be paid to the possibility of using this form of frame construction. Many structural engineers have had no experience of this form, and may be under the impression that it is difficult to design. Advances have, however, been made during recent months, and there are firms in Great Britain capable of designing and fabricating portal frames quickly. Where heavy brick-panel walls are demanded, as in certain vital buildings, their weight can be transferred to the bases of the stanchions, thus rendering it possible to design the portals as fixed-ended This leads to a considerable economy of

steel. These portals can now be designed as quickly as any more orthodox structure.

### Older Buildings.

Many older buildings, multi-storey and single-storey, although having steel beams and internal stanchions, have the ends of the beams or trusses carried on load-bearing walls. In these cases a direct hit on such a wall would have disastrous effects.

Additional resistance to collapse can be given by the provision of stanchions to support the beams if the wall is damaged. The best position for these stanchions depends on the distribution of any heavy loads on the floors, and upon the space available. In general, the best position is not in contact with the wall.

Similar features to that mentioned above are sometimes found in other types of buildings. For instance, in one works inspected, a shop, mainly of reinforced-concrete frame construction, was spanned by long steel joists supporting the roof trusses, the ends of the joists being simply supported on haunches formed on the reinforced-concrete columns. This is clearly objectionable, as a near hit would displace the ends of the joists, with disastrous results. In such a case the joists should be restrained so that they could not move laterally relative to the stanchions.

There are many older types of building which are not satisfactory and which do not lend themselves readily to improvement. In general, as reiterated above, the best steps to be taken are to provide as much continuity and bracing as possible in the load-bearing part of the structure.

### Special Types.

Where there are excessively long-span trusses supporting a roof, as in an erection shed, additional props may be needed owing to the danger of existing stanchions being hit, or a joint or member in the truss being cut by a direct hit.

The principle of continuity explained above should be used as a guide in designing the propping system. Here again the object is to prevent complete collapse. Local over-strain in members is not seriously objectionable, so that it may not be necessary to insert props under every panel-point. Strengthening of joints and additional counter-bracing may further reduce the number of props required.

The greatest care must be taken to secure economy in this work. The structural designer, whose concern it has always been in the past to produce a structure with an adequate factor of safety, will probably not find it easy to consider with equanimity one on the point of collapse. There may well, therefore, be a tendency to over-strengthen the structure.

Additional props and strengthening of the truss as suggested above appear to be preferable to the sand-bagging of existing members.

# CORRESPONDENCE
## ON PAPER PUBLISHED IN
## NOVEMBER 1939 JOURNAL.

---

Paper No. 5213.

## "Application of Experimental Methods to the Design of Clarifiers for Waterworks." †

By Robert Walton and Thomas Dow Key, M. Inst. C.E.

### Correspondence.

**Mr. William Clifford** noted that the problems connected with the arification of Nile water were similar to those encountered in the purifica-on of sewage, and that so far as the removal of suspended solids was con-rned, the object was the same: namely, continuous settlement of the spended solids while the liquid flowed continuously through a tank or ries of tanks.

The form of tank generally favoured by engineers for continuous-flow ttlement, at the time when the Rond Point works were constructed, was ctangular in plan, with an inlet (or inlets) at one end, and an overflow ir at the other; and sometimes with, but more often without, baffle-alls across the middle of the tank.

The results were not unsatisfactory, as long as the volume of water per our flowing through the tank did not exceed from one-twelfth to one-ghth of the capacity of the tank, the amount varying with the character the suspended matter, and with the percentage reduction desired or lerated.

The treatment at Alexandria aimed at a 90-per-cent., or even greater, duction of the suspended matter from the incoming water, and that peared to have been obtained at the Rond Point settling basins, when e volume of flow-through water per hour did not exceed from one-seventh one-eighth of the tank capacity; that was to say, when the ratio of pacity to input per hour was about 7·0 to 8·0. The Authors gave . 26 §) the ratio as 8·9, a figure which did not agree with the tank capacity d daily intake in 1928.

---

† Journal Inst. C.E., vol. 13 (1939–40), p. 21 (November 1939).

§ Page numbers so marked refer to the Paper (Footnote (†) above).—Sec. Inst. E.

The six settling basins at Rond Point had a total capacity of 6 × 985,000 gallons, namely, 5,910,000 gallons, and in 1928 those tanks dealt with 21,000,000 gallons per day, or an average of 875,000 gallons per hour. When all the tanks were in commission, the ratio of capacity to input per hour was $\frac{5,910,000}{875,000} = 6{\cdot}75$, and when one tank was out of commission for de-sludging the ratio became $\frac{4,925,000}{875,000} = 5{\cdot}51$.

Before examining the experimental work, it might be of interest to see, so far as the data allowed, the results of modifying the settling basins at Rond Point, and to compare the results with those obtained by the circular tanks at Siouf. It was not clear to what extent the reduction of suspended solids was affected at the Rond Point works when one tank was out of commission; nor indeed did there appear to be any record of the reduction of suspended solids before the alterations were made.

The percentage reduction of suspended solids in a tank which had been modified by the addition of numerous baffle-walls, was given as 93·2, but whether the ratio of capacity to input per hour was 6·75 or 5·51, was not stated. There was thus no information to show that the introduction of more baffle-walls affected the percentage reduction of the suspended solids or allowed a greater input, without lowering the percentage reduction of suspended solids.

If it were intended to show that the alteration to the structure of the tank materially affected the chemical quality of the effluent, full chemical analysis over a considerable period before and after the alterations under comparable conditions would be required in order to demonstrate that fact, instead of a few figures for clarity and albuminoid nitrogen content.

With a general idea of what the Rond Point modified settling basin could do in effecting the reduction of suspended solids at one or other of two rates of through input, it was possible to compare the results with those obtained with the 110-foot circular tanks at Siouf.

During December 1937 and January 1938, at a period when the amount of suspended matter in the influent was roughly three-fifths of the yearly average, one tank, taking double quantity, gave a reduction of suspended solids of 91 per cent., with an average flow-through of 252,580 gallons per hour, and a ratio of capacity to input per hour of 3·71. With both tanks (Table III, facing p. 48 §) in operation, but with a larger amount of suspended solids present, the percentage reduction of suspended solids was 93·3. In that case the input rate was halved and the ratio of capacity to input per hour was doubled—namely, 7·42.

In other words, the recently-designed circular tank at Siouf gave nearly the same percentage reduction (93·3) of suspended solids from

§ *Ibid.*

e same average liquid as that achieved (93·2) from the Rond Point ttling basins, but the latter had the higher rate of input, the ratios of pacity to input per hour being either 6·75 or 5·5 at Rond Point as compared with 7·42 at Siouf.

The records of the circular tank (Table IV, facing p. 49 §) taking double e usual quantity of liquid were of interest, but unfortunately the results, far as percentage reduction of suspended solids were concerned, were t comparable with any other data in the Paper. The experiment left certain how far the result in that case was affected, (*a*) by the increased te of flow, and (*b*) by the smaller quantity of suspended solids present in e influent.

Mr. Clifford submitted that a better criterion of the elimination of susnded solids would have been to maintain, under normal conditions, one rcular tank at the same ratio of capacity to input per hour as the settling sins at Rond Point, and to increase the flow-through of the other circular nk until the percentage reduction of suspended solids had fallen to the nit beyond which it would be undesirable to increase the flow.

The mechanical collection and hydrostatic removal of the sedimented udge from the Siouf tanks followed the practice successfully employed in wage-purification. It was stated that the de-sludging mechanism worked r 2½ hours per day. It would be of interest to know if that were in one riod, or in several shorter periods totalling 2½ hours per day. Certain periments, made at Wolverhampton some years ago, indicated that very ow continuous stirring of settled sewage-sludge liberated mechanicallyld water and gave a denser sludge.

The object of the experiments with the glass-sided tanks was said to ve been the investigation of whether or not the disposition of the inlet, tlet, and two intermediate walls was conducive to the best possible rculation. Assuming that the Authors meant the best possible circulation r the settlement of suspended solids, there was no indication of any easurements in that connexion. On the other hand, if they meant that a rculation in which any small mass of influent water remained longer in e tank with or without intermediate walls, the average time of stay in e tank did not appear to have been measured. Even if the average time passage through the tank had been determined, however, it would still main to be seen how that affected the settlement of suspended solids. that connexion the time–flow measurements recorded in various parts the Paper were made with liquids for the most part denser than water, e course of which through the tanks was entirely different from that of e water to be measured; and in any case the time of the first appearance the test liquid in the effluent was noted, and not the average time. The fficulty of measuring the average time of passage of water through odel-settlement-tanks led Mr. Clifford many years ago to adopt the term

§ *Ibid.*

"nominal period of retention" to indicate the relation of tank-capacit to the volume of input-liquid in a given time.

The Authors' observation on p. 28 § that small variations of temperatu produced directional changes of flow in the tank, was quite in accord wit the experience of Messrs. Clifford and M. E. D. Windridge.* Differences temperature much less than the 0·2° C. noted by the Authors would ofte determine whether the main direction of flow would be in the upper the lower part of the tank. In the same way, small differences in th quantity of salts in solution and of suspended solids would also produc appreciable directional changes of flow.

The preliminary conclusions (pp. 30–31 §) drawn from the experimen with the glass-sided model-tank were far from convincing:

(*a*) If vertical motion and eddies had to be prevented, the Autho failed to indicate how that was to be accomplished, and gave no measu of the success attained by the use of intermediate baffle-walls.

(*b*) It might be asked what justification there was for the statemen that horizontal eddies with linear velocities up to 10 feet per minute ha no floc-supporting value. That statement might or might not be tru and it would be of interest to know how the linear velocity was determine

(*c*) A settlement tank, as distinct from an indefinitely long channe necessitated water entering the tank at a higher average velocity than th at which it left the tank by the usually wide overflow weir. In oth words, although the masses of water $m_1$ and $m_2$ entering and leaving th tank at any moment were the same, the average velocities, $v_1$ and $v$ were very different. For all practical purposes the whole kinetic energ of the inflowing water, $\frac{m_1 v_1^2}{2}$, had to be dissipated. The engineer migh localize and control the dissipation of the kinetic energy, or, if the ta were sufficiently large, he might leave the dissipation to be effected the haphazard eddies and internal movements in the mass of water in th tank.

The Authors proposed to dissipate the kinetic energy by directing th flow around and over numerous baffle-walls. Apart from any doubtf advantage in the reduction of suspended solids, the introduction of baffl walls was hardly a recommendation for facilitating sludge-removal fro sewage tanks.

The experimental work with the circular model-tank of 2,700 gallo capacity, which was evidently designed to try to find the best conditio for the settlement of suspended solids, left much to be desired. The cruci test for any given set of conditions was surely the difference betwe the quantity of suspended matter going into the tank and that goi

§ *Ibid.*

* "Experiments with Model Tanks." Journal and Proceedings, Inst. Sewa Purification, 1935 (Part I), p. 136.

out. No such measurement appeared to have been made. Many interesting observations of the occurrence of uplift, induced uplift, Borda stream, suction-effect, and other phenomena were recorded, which were significant pointers, rather than any measure of uncontrolled currents in the tank.

The central-feed difficulty with the circular model-tank—a difficulty which had been solved many years ago for sewage settlement tanks—had at least one positive result, in that it diverted the Authors' attention to an alternative method of obtaining suitable conditions for settlement. The mixing tank alongside the circular tank became, in effect, a means for dissipating a large part of the kinetic energy of the influent, and enabled the Authors to have a much larger inlet to the circular tank than they would otherwise have had. For example, if the feed-pipe to the mixing chamber were assumed to be 2 feet in diameter (the size was not given in the Paper) the average velocity when delivering 220,000 gallons per hour would have been 3·1 feet per second, and the mass of water 611 lb. per second, so that the kinetic energy to be dissipated would have been about 2,935 foot-lb. per second. In the absence of other means of dissipating the kinetic energy, that would have taken place in the mixing chamber, which in effect became an energy-dissipating contrivance.

It was unnecessary for the area of the opening between the mixing chambers and the circular tanks to be as limited as in a central feed-pipe, but it could be enlarged to give a relatively low average influent-velocity. The size of that opening, although not stated, was not unimportant: a small opening might well give a high average influent-velocity and produce undesirably high rotational movement in the circular tank. Other things being equal, the orifice—suitably guarded to prevent local high-velocity streams from entering the circular tank—should be as large as possible. Indeed, it was not too much to say that the effectiveness of the tank as a means for the continuous settlement of suspended solids depended upon the area of the inlet to the circular tanks and upon the control of the average influent-velocity across that area.

**Mr. Alexander Ramsay** observed that Bahia Blanca and district was dependent on the flow in a relatively small stream, subject to great fluctuations in flow and in suspended material. Following rains in the catchment, the turbidity commonly increased from 20 parts per million to more than 100 times that value within a few hours, and the necessary treatment had to be effected in settling ponds situated close to the river intake, at a remote, and, after heavy rains, inaccessible point in the country some [illegible]0 miles distant from the filter establishment.

Immediately before entering the settling ponds alumino-ferric was added in a weir-type dosing apparatus, and thorough mixing was assisted by creating turbulence in the flow of the treated water in open channels leading to the two settling ponds. The latter were in parallel and rectangular in form, each measuring 520 feet long and 36 feet wide, with a mean depth of water of 12 feet. The capacity of the ponds was 1,400,000

gallons, and with a flow which might vary from 70,000 to 110,000 gallon per hour, the theoretical retention period varied from 18 to 12 hours.

As originally constructed, each settling pond consisted of an intak chamber, a main sedimentation tank, and an outlet chamber, separate by walls, with only small gate openings of 8 per cent. of the cross-section area of the tank. Under such conditions of operation it was natural th the retention efficiency might be as low as 10 per cent., and that whe turbidities exceeded from 200 to 300 parts per million the water could n be satisfactorily conditioned.

The first modifications to increase the retention efficiency consisted closing the gate openings and converting the division-walls between th inlet chamber and the main tanks into submerged weirs. That increase the retention efficiency to more than 20 per cent. It was considere therefore, that further progress might be made by producing flow-strea in a vertical plane, and baffle-walls were introduced some 40 feet from th inlet chamber. A diffuser-board was also provided in the inlet chamber front of the inlet channel. The diffuser-board appeared to function wi great efficiency, but the efficiency of the baffle-walls did not prove so hig as was anticipated. Temperature-differences were always present, and them, and to the inertia of the main body of water, had to be attribut the relative failure of the baffle-walls. It had always been borne in min that for satisfactory flocculation and sedimentation three stages we essential, namely, mixing, floc-forming, and coagulation, and the insta lation of mechanical flocculators had been under consideration, sin the opinion was held that the second stage was inefficient, even thoug heavily-treated water always gave a good-sized floc. Furthermore, t bulk of the sludge deposited was always found in the inlet chamber, a only up to the first 200 feet length of the main tank, showing that for practical purposes the capacity of the settling tanks was not being exploite

Increasing the aluminium-sulphate dose had also been tried, and it h been found possible to condition water of turbidity 3,000–4,000 parts p million by applying 250 parts of chemical per million. That measu however essential it might be under existing conditions, was costly, n only on account of the first cost of the alum (supplied from Englan but also on account of the necessary alkalinity-corrective treatment wi hydrated lime, in order to render the water non-aggressive to the tru mains leading to the filter establishment.

The information in the Paper that water of high turbidity could conditioned satisfactorily with a moderate use of sulphate of alumina settling-tank arrangements, which would appear to be applicable to t settling tanks of Bahia Blanca, was therefore of great value.

The Authors gave in Table II (p. 34 §) the percentage removal of slu in the different compartments of the modified tanks. Would they g

§ *Ibid.*

comparative data for the tanks as originally designed? What percentage of the total material in suspension was deposited in the first section of the original settling tanks, what was the highest turbidity experienced, and what was the alum dose under those conditions?

The Authors were to be congratulated on the efficient circular clarifier which was evolved after long and laborious experimental work. The difference in design from that proposed by prospective suppliers to the original specifications was outstanding. The Bahia Blanca Waterworks Company had also under consideration a circular clarifier tank, with a central entrance-tower with radial openings, but apparently it was necessary to provide for thorough mixing of the alum dose and for 2 hours' mechanical flocculation before allowing the water to flow to that clarifier.

The Authors did not describe the methods adopted for ensuring a thorough mix, nor did they actually express the opinion that a thorough mix was an essential factor, although the results obtained showed that that had been carefully provided for. Presumably the mechanical stirring apparatus and hydraulic jump included in the experimental tank layout indicated the methods adopted.

**Mr. J. R. Roberts** wished to comment on that part of the Paper which dealt with rectangular settling tanks, in the light of experience gained in the treatment of local waters.

In the treatment of waters in the Gold Coast Colony difficulties as great as those in Egypt were encountered, with the difference that pollution in Egypt was stated to be caused by sewage and proteinic manurial matters, whereas in the Gold Coast pollution was due almost entirely to vegetable matter. The Authors mentioned that the highest dose of aluminium sulphate introduced to the Nile water was some 6 grains per gallon, or 85·8 parts per million, the average dose having been 35·5 parts per million. At Weshiang, where the headworks of the Accra supply were situated, the water of the river Densu at its worst required a dose of from 170 to 180 parts per million, although at other times the dose was as low as 40 parts per million. Bacteriologically, however, the pollution in the Gold Coast waters was much less potentially dangerous than in Egypt, a fact which was reflected in the results of bacteriological examinations referred to in Table I (p. 23 §).

The Authors described the mechanism of the normal reaction when aluminium sulphate was added to water; namely, the precipitation of insoluble gelatinous aluminium hydroxide, the neutralization of electric charges, and the entangling of suspended matter with the aluminium hydroxide. When, however, water contained large amounts of vegetable matter, gallic, tannic, and the so-called "humic" acids, it was considered that that normal reaction took place only in a modified form. In such cases the precipitate was largely contaminated by insoluble compounds of

§ *Ibid.*

aluminium, such as aluminium gallates, tannates, and "humates", and the resulting floc did not seem to be nearly so effective as a normal floc in removing the impurities from the water. Its ragged, dark-coloured appearance was also quite different from that of a normal floc, and it might well be that with such waters it was necessary first to add sufficient aluminium sulphate to precipitate the gallates, tannates, etc., and then to add a further amount to provide the precipitate of aluminium hydroxide which was effective in clarifying the water. That would explain the increased doses of aluminium sulphate which were necessary for efficient coagulation when water was grossly polluted with vegetable organic material, as occurred at the Accra works when the river was falling and was heavily charged with swamp water.

The Authors stressed the necessity for proper conditioning of the floc by bringing individual particles into contact and causing them to roll over each other so that they aggregated into larger and denser particles to facilitate settlement. The sedimentation tanks at most of the Gold Coast works were of the "over and under" type, in which the treated water entered the first compartment at the bottom and left it over the submerged weir formed by the first "over" partition. Much floc collected in that first compartment and it was maintained at a level which was some 6 inches below the top of the submerged wall, the result being that the entering water, charged with newly-formed floc-particles, passed through a large bank of already-formed floc. There was no doubt that that contact between new and old floc resulted in an aggregation of the particles, and much improved sedimentation. That process was similar to that for the conditioning of the floc described by the Authors, but was brought about in a different way. The bank of floc in the first compartment also resulted in a very uniform distribution of the water, and the even distribution of the flow, at least through the first compartment, could readily be gauged by observing the behaviour of the bank of floc.

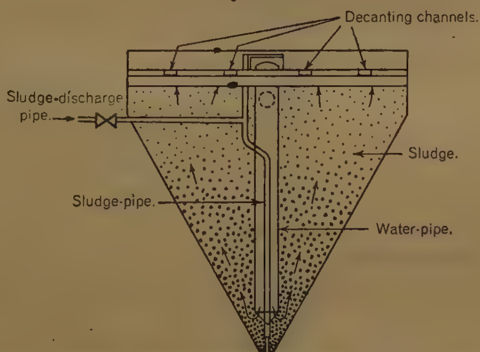
It had been observed, however, that all waters were not equally responsive to that treatment. Although it worked admirably in the case of clay-bearing waters in which a dense heavy floc was produced, the lighter ragged floc previously described, which was characteristic of heavy organic pollution, did not aggregate and "bank up" so well.

The Authors stated that with their improved system of baffling, 93·2 per-cent. removal of suspended matter was achieved. When the "over and under" tanks were operated so as to provide a theoretical retention period of 8 hours, results equal to that were obtained, but when the tanks were operating at higher rates of flow, the percentage removal was lowered. For example, at one works when the rate of flow was 400,000 gallons per day, with a retention period of 7½ hours, the removal effected was 94 per cent. At a rate of flow of 1,000,000 gallons per day, however, and with a retention period of 3 hours, some 89 per cent. was removed. Those figures were gravimetric estimates similar to those of the Authors.

Although the sedimentation tanks in the Gold Coast Colony were reasonably efficient, greater efficiency could undoubtedly be obtained by introducing the system of baffling recommended by the Authors.

**Mr. R. C. S. Walters** observed that some years ago he was associated with experimental work on humus tanks at Cheltenham. The effluent was admitted centrally at the bottom of conical tanks about 20 feet deep, and flowed upwards to a circumferential channel. Observations were made, by tintometer, of the variation in colour of the effluent with the amount of oxygen absorbed. A close relationship existed between the colour and the amount of oxygen, but it was noticed generally that when the tanks were almost full of humus, the effluent improved enormously. In recent years, more light had been thrown on the subject, and the improvement in the effluent could be attributed to penetrating the blanket of suspended sludge matter.

*Fig. 20.*




Mr. Walters was attracted to that basic principle when it was introduced at the Hyderabad waterworks for the preliminary purification of a stream for waterworks purposes in India in 1934. The object of the installation was to reduce the work in the sand filters and to reduce the cost of alumina, by bringing, for example, a water capable of forming a small floc into contact with a ready-made colloidal blanket, quietly, naturally, without baffles, and without mechanical agitation.

The water was introduced into conical tanks at the bottom of a central pillar, and flowed upward at a decreasing velocity through the solid and sludge matter already in suspension. Special precautions were, however, taken to ensure that the water should be collected, not at the periphery, but by means of decanting channels carried over the top of the tank (*Fig. 20*); that the colloidal matter should not be emptied until a predetermined amount had been formed in the tank; and that the amount of colloidal matter emptied from the tank should be limited.

The following points were of interest: (*a*) mechanical mixers were

eliminated; (*b*) the blanket of several feet of sludge automatically took care of variations in the character of the water; (*c*) the top few feet of water being clear, wind-effect did not arise; and (*d*) there were no baffles.

It was probable that the conditions of an ordinary upland-area stream in England were very different from those of the silt-laden Nile, which was capable of forming an easy floc. It would, however, seem from the Authors' remarks that they would not favour upward flow in conical tanks for every type of water, although it was doubtful whether the Authors had investigated sufficiently not only that particular type of tank, but also a sufficient number of streams.

**Mr. Bernard Whitteron** observed that there was some inconsistency in the figures given regarding the experiments: taking the figures of flow and capacity given in Table IV (facing p. 49 §), the flow and capacity in the model would appear to be as calculated in Table V, whereas the Authors gave a flow of about double that calculated value, and also quoted a theoretical retention time of 3·4 hours.

TABLE V.

| | Full-scale clarifier (from Table IV). | Model-clarifier. | | |
|---|---|---|---|---|
| | | As given. | As calculated. | Scale of mode |
| Diameter: feet . . . . . . | 110 | 16·4 | — | 1 : 6·7 |
| Capacity: gallons . . . . | 935,000 | 2,700 | 3,100 | $1 : (6{\cdot}7)^3$ |
| Flow: gallons per hour . . . | 220,000 | 3,600 | 1,900 | $1 : (6{\cdot}7)^{2\cdot}$ |
| Theoretical retention-time: hours | 4·25 | 3·4 | 1·63 | $1 : (6{\cdot}7)^{\frac{1}{2}}$ |

The inconsistencies in the scale figures would seem to invalidate many of the conclusions drawn from the experimental work.

Radial-flow tanks in normal circumstances were accepted as being efficient, and Dr.-Ing. Max. Prüss, describing a 223-foot diameter, radial-flow tank for clarifying water of the river Emscher, quoted * a removal of 96 per cent. of suspended matter after 1 hour's theoretical retention (equivalent to 100 per cent. overload on the normal rate of working).

It was surprising, after the disappointing results of the first three groups of experiments, that the circular form of clarifier should have been retained, the only reason given being its cheapness and simplicity of construction.

Mr. Whitteron suggested that the terms "retention period" and "detention period", to which the Authors objected, were misleading only in so far as they were incomplete; the term "theoretical retention period" was exact, and for written description was less cumbersome than

§ *Ibid.*

* "Sedimentation on a Large Scale." The Surveyor, vol. xcii (1937), p. 29 (September 10, 1937).

the suggested term " capacity output per hour ", or the expression " Ratio : capacity input " used in Table IV (facing p. 49 §).

It was stated that one of the principles laid down was that the settling basins should precipitate 90 per cent. or more of the suspended matter. The important factor was that the effluent should not fall below a certain absolute standard for any anticipated condition of influent. In fact, Table III (facing p. 48 §) showed effluents with 24·7 parts and 23·1 parts per million respectively at periods when the turbidity of incoming water was 93·7 and 704·7 parts per million. Mr. Whitteron considered that it would be preferable to lay down an absolute standard of effluent—for example, that the suspended matter should not exceed 50 parts per million and should have a clarity of more than 30 centimetres. The recognition of some such standard, however widely varied for individual conditions, was very necessary, and the Authors' recommendation as to such a standard would be valuable. Beyond a certain point the clarification was performed with infinitely greater ease by filters, and just as it was wasteful to attempt clarification beyond that point, so it was equally important that the amount of suspended matter should not be greater than the filters could deal with. The stressing of clarifier efficiency tended to obscure that fundamental requirement.

The Authors stated that their original preference for a rotary motion was because it would be practically uninfluenced by wind- and temperature-effects. The reason for that was not obvious. After abandoning the experiments with radial flow, they reiterated the conviction that satisfactory results could be obtained " provided that the contents could be induced to follow some kind of progressively narrowing or widening helicoidal motion about the centre." It was unfortunate that the Paper did not elaborate " ancillary details connected with the precise way in which the water flows . . . " Gyratory flow was in principle incompatible with uniformity of flow. The results might be held to be sufficient justification for the principles which were embodied in the design, but a reasoned analysis of them would give added conviction.

**The Authors' reply had not been received at the time of going to Press.**—Sec. Inst. C.E.

§ *Ibid.*

# CORRESPONDENCE
## ON PAPER PUBLISHED IN
## DECEMBER 1939 JOURNAL.

---

Paper No. 5219.

## "The Analysis of Flow in Networks of Pipes." †

By Ronald James Cornish, M.Sc., Assoc. M. Inst. C.E.

### Correspondence.

**Mr. J. R. Daymond** observed that the principle of successive corrections as given in the Paper was finding a wide field of application in problems where exact solutions were not needed, or where such solutions were troublesome to find. The method was particularly valuable in pipe-line and allied problems, where in any but the simplest layout of pipes algebraic solutions were very tedious. In fact, apart from the very simple cases of pipes in series (that was to say, one pipe-line), pipes in parallel (namely, the " Cross-Over System " *), and some three-reservoir problems (for $n = 2$, only), direct algebraic solutions were not possible.

In the restricted field where direct solutions were possible in pipe-networks, there sometimes existed the advantage of obtaining information which was hidden in approximate methods of solution. The " Cross-Over System " provided a good example of that, and it was quickly shown how an algebraic solution gave information of value. He would suppose that an existing pipe of length $L$ and constant $\alpha$ connected two reservoirs A and B, the flow being from A to B. The pipe was duplicated for part of its length by an added main of length $l$ and constant $\alpha_1$. Neglecting velocity heads and considering friction losses only‡, it was readily shown if $\phi = l/L$, and $(\zeta - 1)^n = \alpha/\alpha_1$, that

$$\phi = \frac{\zeta^n(\kappa^n - 1)}{\kappa^n(\zeta^n - 1)},$$

where $\kappa = Q_2/Q_1$, $Q_2$ denoting the flow to B through the existing and

† Journal Inst. C.E., vol. 13 (1939–40), p. 147 (December 1939).

* F. W. Macaulay, " Cross Connections on the Elan Aqueduct of the Birmingham Corporation Waterworks." Minutes of Proceedings Inst. C.E., vol. ccxi (1920–21 part 1), p. 30.

‡ Those assumptions for head and losses would be maintained throughout the following observations.—J. R. D.

added pipes and $Q_1$ denoting the flow through the existing pipe only. It might be noted that $\phi \not> 1$, and therefore $\kappa \not> \zeta$.

If a main were to be duplicated in a given number of years to cater for an annual increase in supply, the length to be added at any time could be immediately derived from the above equation. Also, it could be easily shown that, for a given value of $\zeta$, $\kappa$ increased rapidly with $\phi$. Hence, as complete duplication was approached ($\phi \rightarrow 1$), the length of pipe to be added to meet a given increase in supply became smaller. Therefore, since additions to the existing pipe might be made in any order, it followed that, to equalize annual costs, the most costly portions per unit length should be duplicated last.

Probably the greatest value of the method given in the Paper was in the rapid convergence obtained. Even in problems amenable to direct algebraic solution, it was often quicker and accurate enough to use the method of successive corrections. Problems in three-reservoir work provided good examples to support that contention.

The improvements on the pioneer work of Professor Hardy Cross given by Mr. Cornish shortened the work considerably, since for each approximation consistency of head-drop around the circuit considered was always ensured. It might be suggested that the work might be shortened, whenever possible, by deriving beforehand the direction of flow in each pipe. For example, in *Fig. 1* (p. 149 §), it was evident by inspection, and before applying any analysis, that the flow at joint B would be A to B, B to C, and B to E. Similarly, the flow direction at E was immediately established. Such a method was a very useful preliminary to solving problems in three-reservoir work, and to students, in particular, it might be worthy of further mention.

Mr. Daymond would consider three pipes AJ, BJ, and CJ, joining three reservoirs A, B, and C, to J, and would assume that all essential data were known except the head at J and the rate of flow in each pipe. In general, there would be doubt as to the direction of flow in one pipe, BJ for example. He would assume that there was no flow in that pipe; that was to say, that the head at J was equal to the head at B. For the assumed conditions he would denote the flow in AJ and JC by $Q_1$ and $Q_2$ respectively.

It then followed that, if $Q_1 = Q_2$, there was no flow in JB;
$Q_1 > Q_2$, the flow would be from J to B;
and $Q_1 < Q_2$, the flow would be from B to J.

Following that preliminary investigation, a solution was then obtainable by the method given in the Paper.

It was suspected that the clarity and full import of the proof given in Appendix I (p. 153 §) had been somewhat sacrificed to brevity. That was

---

§ Page numbers so marked refer to the Paper (Journal Inst. C.E., vol. 13 (1939–40), p. 147 (December 1939)).—Sec. Inst. C.E.

regrettable, because one or two important points, although referred to an[d] utilized in the Paper, were obscured in the proof. They were deservin[g] of greater prominence, and the following discussion might be of interest.

He would suppose that $m + p$ pipes met at a junction J. He woul[d] assume that in any one of the $m$ pipes there was a flow $Q_m$ towards [J] and that in any one of the $p$ pipes there was a flow $Q_p$ from J. If at th[e] ends of those two pipes remote from J the heads were either assumed o[r] known to be $H_m$ and $H_p$ respectively, then, using the formula of th[e] Paper, it was possible to write

$$\left.\begin{aligned} H_m - H &= \alpha l_m Q_m^n \\ \text{and} \quad H - H_p &= \alpha l_p Q_p^n \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{i})^*$$

where $H$ denoted the head at J and $l_m$ and $l_p$ denoted the lengths of th[e] respective pipes.

There could be no accumulation of flow at J, and if $H$ were correctl[y] chosen, the total flow to J, $\Sigma Q_m$, would equal the total flow from J, $\Sigma Q_p$. For the assumed conditions, however, he would suppose that there was a[n] excess $\Sigma\delta Q$ of inflow over outflow at J, so that

$$\Sigma\delta Q = \Sigma Q_m - \Sigma Q_p . \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{ii})$$

From (i) and (ii) it was seen that $\Sigma\delta Q$ was a function of $H$, namely

$$\Sigma\delta Q = f(H) . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{iii})$$

If, by increasing $H$ to $H+\delta H$, continuity of flow ($\Sigma\delta Q = 0$) was ensure[d] at J, then, in (iii),

$$0 = f(H+\delta H).$$

Using Taylor's expansion, and making the usual assumptions for it[s] validity, led to

$$0 = f(H) + \delta H f'(H) + \ldots$$

Subtracting (iii) from the latter expression gave

$$\Sigma\delta Q = -\delta H f'(H).$$

Differentiating (i) and substituting in the above equation,

$$\Sigma\delta Q = \frac{\delta H}{n}\left(\sum\frac{Q_m}{H_m - H} + \sum\frac{Q_p}{H - H_p}\right) \quad . \quad . \quad . \quad (\text{iv})$$

The terms to be summed in (iv) were all positive, and therefore th[e] summation could be taken over the $m + p$ pipes without regard to sig[n] (as noted in (4), p. 148 §). If $Q$ denoted the flow in any pipe in whic[h]

---

* It was evident there would be no flow in any pipe where $H_m = H$, or wher[e] $H_p = H$; therefore that pipe could be omitted from the particular balance, as men[-]tioned in paragraph (2) on p. 150 §.—J. R. D.

§ *Ibid.*

the head lost was $h$, then from (iv), after making the necessary substitutions, and rearranging,

$$\delta h = \frac{\Sigma \delta Q}{\Sigma(Q/nh)},$$

the result given in the Paper.

An interesting variation from the case of steady flow given in the Paper, and one in which the conditions corresponded more closely to those in a town's network, was to assume a fixed rate of abstraction, say $q$, from the pipe per unit length. In a pipe of length $l$ the total abstraction would be at the rate $ql = Q_0$. If the flow at a junction were taken as $Q$, the flow in the pipe at the other end would be $Q + Q_0$. In such a case it could be easily shown that the loss of head, $h$, was given by

$$h = \frac{\alpha l}{(n+1)Q_0}((Q+Q_0)^{n+1} - Q^{n+1}), \quad . \quad . \quad . \quad (\text{v})$$

or

$$h = f(Q),$$

since $Q_0$ was a constant.

If $\Sigma Q_c$ and $\Sigma Q_a$ were the respective total clockwise and anti-clockwise flows in a circuit, it followed that

$$\Sigma \delta h = \Sigma f(Q_c) - \Sigma f(Q_a) \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{vi})$$

If an anti-clockwise flow $\delta Q$ provided the balance in the circuit ($\Sigma \delta h = 0$), then

$$0 = \Sigma f(Q_c - \delta Q) - \Sigma f(Q_a + \delta Q).$$

Expanding that expression, assuming $\delta Q$ small, and afterwards subtracting it from (vi), led to

$$\Sigma \delta h = \delta Q(\Sigma f'(Q_c) + \Sigma f'(Q_a)) \quad . \quad . \quad . \quad . \quad . \quad (\text{vii})$$

From (vii), it was seen that the summation, after differentiation, might be taken over all the circuit irrespective of sign (see (4), p. 152 §) provided that regard was paid to the sign of $\delta h$ in (vi) (see (3), p. 151 §). Hence, (vii) might be written

$$\Sigma \delta h = \delta Q \Sigma f'(Q).$$

Differentiating (v) and substituting in the above equation,

$$\Sigma \delta h = \delta Q \Sigma \alpha l((Q_0 + Q)^n - Q^n)/Q_0 \quad . \quad . \quad . \quad . \quad (\text{viii})$$

If it were required to use heads, instead of flows, (viii) could be easily transformed to read

$$\Sigma \delta h = \delta Q \Sigma (h_1 - h_2)/Q_0,$$

where $h_1$ denoted the head lost for a flow $Q_0 + Q$ and $h_2$ the head lost for a flow $Q$. Applying methods I or II of the Paper, problems in variable flow might be readily solved as suggested in the above example.

**The Author,** in reply, thanked Mr. Daymond for his observations, which added considerably to the value of the Paper; the extension to the

§ *Ibid.*

case of abstraction at a fixed rate was of particular interest. He agreed that it was advantageous to determine the direction of flow in each pipe when that could be done by inspection; an adjustment to the head at a junction sometimes caused an obviously incorrect reversal of flow in a pipe, in which case it was advantageous to adjust the heads at both ends of that pipe at the same time. Professor Hardy Cross had stated * that "the first approximations need not be made very formally. With some experience it is possible to nearly adjust a network at once by a little judicious guessing."

* Analysis of Flow in Network of Conduits or Conductors." University of Illinois Engineering Experiment Station, Bulletin No. 286 (November 1936), p. 29.

# CORRESPONDENCE
## ON PAPERS PUBLISHED IN
## JANUARY 1940 JOURNAL

### Paper No. 5229

## "The Haifa–Baghdad Road."†

By LIEUTENANT-COLONEL RAWDON BRIGGS, D.S.O., M.C., R.E.

### Correspondence.

**Mr. A. de Leeuw**, of Jerusalem, observed that everyone acquainted with conditions in the Near East would agree that a difficult piece of work had been carried out in a very short period.

He was most impressed by the use of modern machinery for executing the work. The Author stated on p. 200 §, under "The Plant, Machinery and Transport": "Owing to the difficulty of maintaining a large labour force in the desert, machinery was used where possible in preference to hand labour. In wartime a large labour force on a road is most vulnerable." That in itself was an adequate reason for the use of machinery. The Author then went on to say, that it was his experience that in Transjordan, where a labourer's wages were 3*s*. (15 *piastres*) for a 10-hour day, including

† Journal Inst. C.E., vol. 13 (1939–40), p. 195 (January 1940).

§ Page numbers so marked refer to the Paper (Footnote (†) above).—SEC. INST. C.E.

the cost of his water and accommodation, a considerable economy resulted from the use of machinery. In some cases the saving was as much as 50 per cent. Mr. de Leeuw's experience at the Dead Sea, where big works were carried out by Palestine Potash, Ltd., and where the whole year round more than a thousand Arabs were employed, was that during difficult times in Palestine—namely, political disturbances, breakdowns of transport, and so on—it proved to be advisable to use machinery and to employ as few men as possible, as it was often impossible to get sufficient men. It was not found, however, that the use of machinery resulted in any economy. In 1938 large flood-protection dams had to be built at the south end of the Dead Sea. The time for execution was limited. The work was therefore carried out by employing, in addition to from four hundred to five hundred Arab labourers, one dragline and twenty tractors and scrapers. The use of those machines enabled the work to be completed in time before the winter rains. An Arab labourer was paid 2*s.* (10 *piastres*) for an 8-hour day. Including the cost of water, accommodation, and medical service, and adding 2½ *piastres* for another 2 hours' labour, the cost was found to be 3*s.* (15 *piastres*) for a 10-hour day, the same as that quoted by the Author. The unit price with machinery, however, was practically the same as with hand-labour. The work consisted of building earth dams, on an average 2—2½ metres high, which were constructed by taking the necessary earth from nearby. The work as described by the Author on p. 206 § would cost the same, whether carried out by Arab labour or by machinery under the conditions obtaining at the Dead Sea. Mr. de Leeuw would, therefore, be very grateful to the Author if he would give some more information on that subject.

On p. 213 § the Author stated that in November 1937 a heavy rainfall occurred. During that month there were very heavy floods at the south end of the Dead Sea, and in that case also the local Arabs said that that was the highest flood that they had ever experienced. That source of information was unreliable, however, as they seemed to make that remark after every flood. The same difficulties were encountered as described by the Author. No reliable data were available and erosion-marks were misleading. Heavy floods, which occurred infrequently and were generally of very short duration, would leave in most cases marks which disappeared after a short time. To estimate floods in the Near East by the observation of erosion-marks would not serve as a reliable guide to the floods that might be expected. It was, therefore, not surprising that the flood heights observed were quite twice those estimated by the engineers from the observation of erosion-marks on the banks. Those erosion-marks would give only an indication of smaller floods which occurred at more frequent intervals and were mostly of longer duration.

The day after the big floods of November 1937, the flood-marks were

§ *Ibid.*

observed and the discharge was calculated. Those flood-marks had since disappeared entirely. The calculation of the discharge was based on the principles laid down by Mr. B. D. Richards, M. Inst. C.E.*

**Mr. A. M. Hamilton** accompanied the Author on the reconnaissance for the Haifa–Baghdad road, and, in the early stages, gave advice on the question of road machinery based on his own previous experience of that type of work in Iraq. Whilst he had had no connexion with the construction work since it began, he was fully aware of all the difficulties and problems that had faced the Author and his road staff, and was surprised at the rapid and excellent progress that had been made.

Amongst the difficulties encountered (which the Author modestly did not stress in his Paper), Mr. Hamilton referred to the very serious unrest in all the Arab countries following the large-scale Jewish immigration which had led to the fear of a Zionist domination of Arab lands and peoples. That made it doubtful, at first, whether the necessary assistance and goodwill of the Arabs of Transjordan would be forthcoming. Moreover, even when the required labour had been found, the problem of providing them with food and water in the middle of a great desert could have been no small one.

In his Paper, the Author had naturally given almost all his attention to the sections of the road where his own work was proceeding. The following description of the route as a whole might, therefore, be of interest, for Mr. Hamilton believed that the Haifa–Baghdad road would form the western end of a future great trunk-road over Asia, and from an engineering and a transport point of view the other sections were equally important.

The Author referred only briefly to the Haifa–Jordan section of the road, which ran *via* Nazareth to Lake Tiberias. It was, as the Author stated, already a very good road: it had, however, to be classed as a mountain road. It had to climb to no less than 1,000 feet above sea-level in order to cross the hills of Galilee, and it then fell 1,500 feet to the shores of Lake Tiberias by rather steep gradients, with much twisting and cornering.

A considerably more direct route, with no mountain range to cross, was followed by the existing railway line which utilized the plain of Esdraelon (said to be the battlefield of Armageddon), crossed a low pass of only 270 feet altitude, and descended into the Jordan valley *via* the Nahr Jalud near the town of Baisan. Mr. Hamilton considered that that would be the best alignment to use; it passed through the natural gateway from Palestine to the east. The Turks had used it as their chief road to Damascus and the north, and, both by road and by rail, it was the main artery of their formidable resistance in the Palestine campaign.

* "Flood-Hydrographs." Journal Inst. C.E., vol. 5 (1936–37), p. 405 (March 1937).

"Further Note on Flood-Hydrographs." *Ibid.*, vol. 11 (1938–39), p. 585 (April 1939).

In 1937 the greater part of the Haifa–Baisan–Jordan route had already been completed by the Palestine Public Works Department and practically the only section requiring reconstruction was that in the Jordan Valley, where the surface was still poor and there were some old and inadequate Turkish bridges. With the necessary regrading and improvement, it would be easily possible for the heavy vehicle-trains of the types used by the Iraq Petroleum Company and the Nairn Transport Company to use that road, whereas they would have the greatest difficulty in climbing the passes and negotiating the zigzags of the Galilee road. The shorter mileage *via* Baisan was also a very important consideration in a trunk road.

A long climb was necessary after crossing the Jordan (elevation —877 feet), for the road had to rise to the top of the Transjordan plateau (elevation +2,500 feet), which terminated there in a rugged escarpment cut by many deep *wadis*. The road followed the Wadi el Arab, which was probably the best route; but the way was difficult, and in order to make the defile as easily surmountable as possible there would need to be successive widenings and reconstructions over a period of some years as the growing traffic demanded it. Various improvements specified by the Author were being carried out by the Transjordan Public Works Department, and those would, it was hoped, be a first step towards bringing the road up to a really modern standard of width, gradient, superelevation, and curvature; much improvement was essential if the vehicle-trains mentioned, which were so important in economical desert transport, were to be able to reach the wharfs at Haifa.

It was to be noted that, for every ton conveyed, large vehicles were considerably the cheapest in running cost; they could, moreover, transport, at a high speed, classes of goods and merchandise that ordinary lorries could not convey at all. In fact, only by using heavy and long vehicle-trains, could railway rates of transport be approached.

The Irbid–Mafraq section traversed the plateau and passed through a fertile catchment-area much intersected with stream beds. A particularly interesting feature of the reconnaissance in that section (not mentioned in the Paper) was the use of aerial photography to secure a better route. After a good deal of exploration had been carried out by car (fortunately it was the dry season, and so it was possible to drive almost anywhere) a preliminary road-line was chosen and fixed relative to various landmarks which could be easily recognized from the air. A large number of vertical exposures were then taken by the Royal Air Force at constant altitude, flying in the early morning and late afternoon when the shadows of hills and *wadis* were most prominent. About 150 individual pictures were taken, and they matched together almost perfectly into a mosaic, thus forming a map quite suitable for checking the new alignment.

The Author mentioned the record floods in the Irbid-Mafraq section in 1938, which actually proved to be a help, in that they enabled bridges and culverts to be determined with a margin of safety not otherwise

possible; he regretted that the floods did not extend to the desert area beyond Mafraq as well, so that the maximum *wadi*-flows might hav been recorded there also. That regret was endorsed by Mr. Hamilton whose experience in Iraq proved that it was not the hill torrent which wa most to be feared by the road engineer, but the usually dry *wadi* on th wide open desert where rain was but seldom known. The occasiona cloudbursts falling on to an immense catchment-area could produce mighty bore of water in a few hours which would engulf and wreck an car or lorry unfortunate enough to be trapped in such depressions; 4 hours later the torrent might be spent and the *wadi* bed almost dry again but any inadequate bridge would have been either completely washed awa or left isolated with its approaches vanished and its piers undermined. Th Iraq Public Works Department practice in such cases was, as the result o experience, either to provide a bridge of ample span or spans, or, if buildin stone or concrete gravel were available, to provide an equally ample over and-under causeway well ramped or protected on the downstream side t prevent scour. If neither method seemed practicable, the situation was lef entirely alone, as it was no use temporizing or guessing at the flow withou accurate knowledge of flood maxima over several years. Moreover, an alteration to the natural drainage-lines owing to the building of raise roadways had to be very carefully considered.

With regard to the passage eastward from Mafraq into the dreadfu lava belt, Mr. Hamilton heartily endorsed the Author's apt descriptio that it "is one of the most desolate and forbidding areas on the face of th earth." It was amazing that either a pipe-line or a road could be buil there. The old volcanoes that produced the seas of lava were still to b seen in the Jebel Asfar and Jebel Aritain and the Ashquaf Ridge, whos crater areas were crossed by the road-line. The road crossed that ridg at an elevation of 3,168 feet.

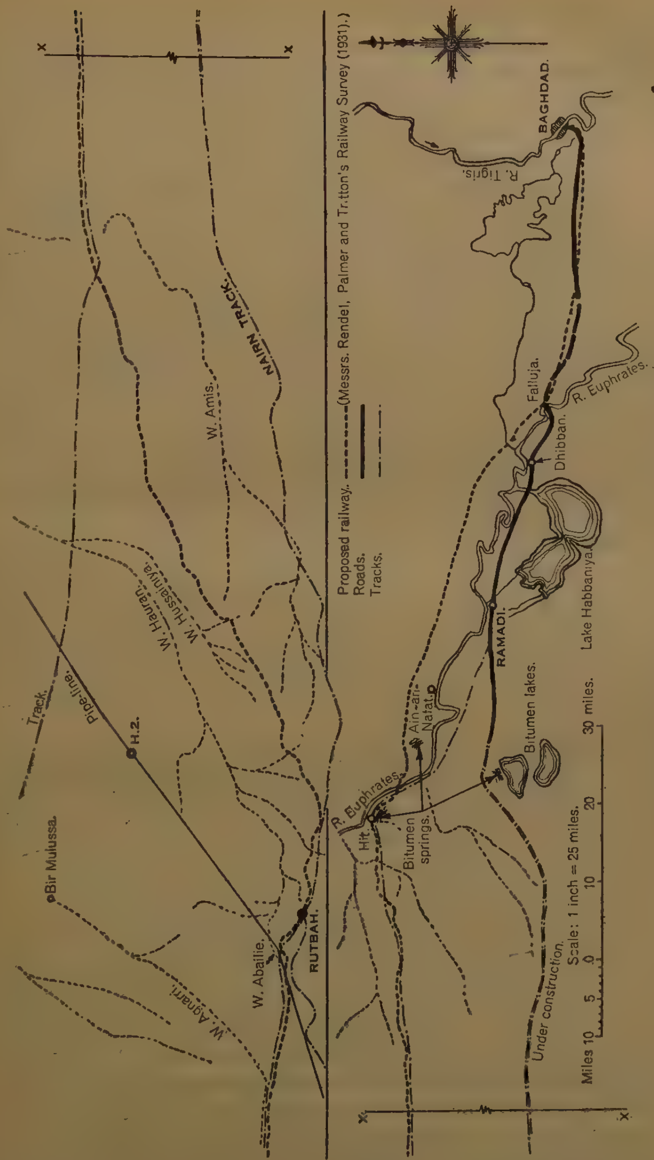
Road-making stone was, of course, plentiful, but was very tough t crush. No "rooting" machinery could ever have had a harder task tha the ripping out of the road subgrade.

The lava area extended for 100 miles, and then gave way to the leve open "steppe" formation which stretched as far as the Iraq-Trans jordan frontier. It appeared to be a dry earthy desert with flint stone scattered over the surface, but rock was not generally to be seen and i was somewhat surprising that the elevating graders found it to be quit difficult work. Nevertheless, the total time, 6 months, was remarkabl short considering the length of the section and the few machines used.

The Author described the work only as far as the Iraq frontier. Tha point was considerably less than half-way to Baghdad, the distance fro Haifa to the frontier being 275 miles, whilst from Haifa to Baghdad th distance was 625 miles; any description of the very extensive Ira sections of the road (*Fig. 3*) could refer only to a few of the main problems

Within Iraq the road began to drop very gradually from the plateau

*Fig. 3.*




THE IRAQ SECTION OF THE HAIFA–BAGHDAD ROAD.

and the fall continued steadily to Ramadi, on the right bank of the Euphrates, the total distance from the frontier being some 280 miles.

Between the desert fort of Rutbah and Ramadi, a distance of about 193 miles, aligning and formation was proceeding in 1937, and a partly-completed road led out of Ramadi for about 60 miles. Mr. Hamilton did

not know whether the whole section was yet finished. The stone had to be transported considerable distances, and the bitumen used came from the bitumen lakes, shown in *Fig. 3*. The specification for that road was not up to the standard laid down for the Transjordan section, and in certain parts the surface was reported to have suffered from traffic and weather.

The special importance of the Rutbah–Ramadi section lay in the fact that two desert traffic-lines joined at Rutbah, and then proceeded by the same road onwards to Baghdad. One branch was from Haifa through British mandated territory as described, the other through Syria *via* Beirut and Damascus. The latter was still the chief route, for it was used by the Nairn Transport Company and various Baghdad motor proprietors. It passed to the northward of both the lava belt and the Jordan Valley depression, but before reaching the coast it had a formidable climb over the high range of the Lebanon mountains, where, once again, vehicle-trains could not, as yet, be used. For passengers, that meant a mass transference at Damascus to fleets of smaller cars.

Mr. Hamilton wished to point out, incidentally, that the Nairn brothers were New Zealanders, and not Australians as stated in the Paper.

From Ramadi down the Euphrates to Falluja the existing road was, on the whole, of good standard. One difficulty, however, arose in dealing with the problem of the Habbaniyah overflow. In times of flood the waters of the Euphrates flowed over the road to fill the Habbaniyah lake. Could the Author state what was being done about that section of the road to avoid the long detour around the Habbaniyah lake when the river broke banks?

At Falluja, the Euphrates was crossed by a modern bridge of five 180-foot steel spans on concrete caissons, completed in 1931. Thence to Baghdad the road traversed a considerable distance over the open desert, the surface gypsum forming a natural matrix fit for all-weather use; it had, however, a tendency to corrugate, and speed was limited.

In Baghdad two fine new bridges over the Tigris, constructed by Messrs. Holloway Bros., Ltd., had just been completed, and those would terminate a chain of highway construction covering a distance of no less than 625 miles from the Mediterranean sea. East of the Tigris two main Asian highways ran on. One passed through Khanaqin to Teheran; the other proceeded to Azerbaijan *via* the highlands of Kurdistan. There would, in due course, be better roads to the Persian Gulf region, and to India.

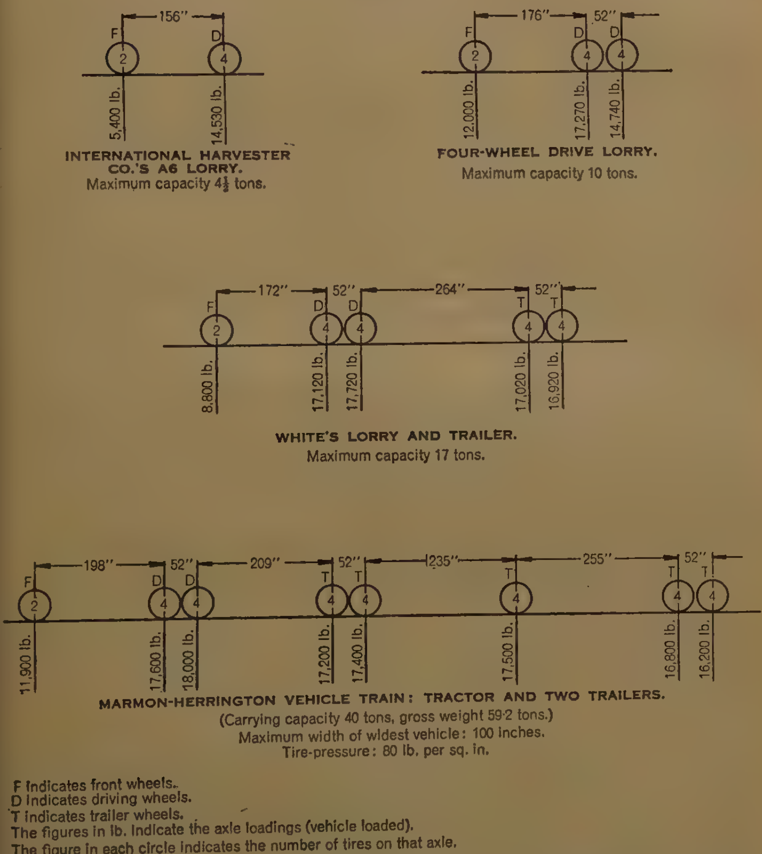
It would appear from what was stated on p. 199 § that there was a restriction on the types of vehicles which might use some, if not all, of the Transjordan bridges. If the bridges were so light in construction as to make that essential, it was a source of danger, because rules were almost impossible to enforce upon such an immense length of road, and

[1] A. M. Hamilton, "Road Through Kurdistan." Faber & Faber, Ltd., London, 1937.

§ *Ibid.*

Arab drivers were a law unto themselves as to where they drove and the risks they chose to take. The object of providing "medium-load" instead of "heavy-load" bridges was, no doubt, to save money on that item and thus to enable more road work to be carried out. The Author mentioned that they might be strengthened later, but in Mr. Hamilton's

*Figs. 4.*




Wheel-Loadings for the Heaviest Vehicles at Present Used on the Desert Route.

opinion that should be done at once. With the comparatively short spans involved such strengthening would be quite an easy matter, and in most cases would merely mean one or two extra joists per bridge. *Figs. 4* showed the axle-spacings and loadings of the heaviest vehicle-combinations using the desert route, and Mr. Hamilton suggested that the bridges might be redesigned to be suitable for the heaviest of those, or, better still,

should be made capable of carrying the Ministry of Transport standard loading.

It would be seen that the generally-accepted idea that desert roads carried lighter vehicles than European roads was not correct for the desert under discussion; the vehicles were just as heavy and speed-impact effects were just as great, whilst the necessity for the heavier vehicles was even more important for successful and cheap desert transport. Large vehicle-trains could carry goods at less than half the cost of single lorries.

The Author had brought together the most modern types of road-making machines to be found anywhere in the world. That had speeded up the work and had eliminated the necessity for large labour gangs. It was, in fact, one of the finest batteries of road-construction plant ever used. The plant represented a very considerable item in the total cost of the work, but the reduction in the unit earth-work and road-making costs by the use of such machinery—as opposed to the old pick-and-shovel style—was probably even more than the 50 per cent. that the Author mentioned, especially when time saved and earlier returns were taken into account. He believed that much the same conclusion had been reached by the Public Works Department of New Zealand in regard to mechanized road-construction.

Mr. Hamilton emphasized that British makers of such road machinery should provide adequate spare parts and service personnel if they were to compete successfully with American firms in that new field. It was chiefly to the Americans that the Author had had to turn, rather reluctantly, for the greater part of the machinery used.

Finally, it should be stressed that the Haifa-Baghdad road should be visualized in three main sections:

| | | |
|---|---|---|
| Section 1, Haifa—H.4, | 232 | miles. |
| Section 2, H.4—M.70 (west of Ramadi, approximately XX in *Fig. 3*), | 252 | ,, |
| Section 3, M.70 (west of Ramadi)—Baghdad, | 141 | ,, |
| Total | 625 | ,, |

Sections 1 and 3 were being worked upon, and everything possible should be done to make them into first-class roads to carry the very heaviest classes of vehicles that were ever likely to pass over them. Section 2, the longest section, was, on the whole, a fair natural road capable of carrying (for about 50 weeks in the year) the heaviest classes of vehicles at a high speed with little difficulty, although there might be the inconvenience of dust. It was Mr. Hamilton's opinion that the barest minimum of work should be carried out on Section 2 until Sections 1 and 3 had been brought up fully to the final standard of bridging, alignment, curvature, width, etc. That work had been done on some 45 miles of Section 2 was in a sense unfortunate, in view of the crying need for the raising of the standard of parts of Sections 1 and 3.

It was also Mr. Hamilton's opinion that the provision of a telephone line along the whole length of the central desert portion of road, together with police posts at intervals of 40 miles, was an even more urgent necessity than was the improvement of that part of the road itself.

**Mr. Samuel McConnel** observed that the Paper appeared to deal with only about one-third of the distance between Haifa and Baghdad, and the cost of the road was about £2,500 per mile.

One of the first questions to be considered before such a road could be constructed was the probable volume of traffic, which in that case was likely to be very small, unless the road was being constructed for strategic purposes. The use of American road plant was becoming fairly general with the reduced costs of operation of diesel tractors, but there were still large areas of Africa in which road construction could be more economically carried out by hand-labour, although the use of light graders and auto-patrols for maintenance purposes could be justified. In the construction of the Haifa–Baghdad road what would be considered in South Africa and Australia as a very heavy and expensive bituminous surfacing had been employed. About 10 years ago the Victorian Country Roads Board had carried out a large amount of experimental work on inexpensive surface treatments and carpets, and since that date a great deal of progress had been made. During a recent visit to South Africa Mr. McConnel had been informed by a number of experienced engineers that a pre-mix from 1½ to 2 inches thick laid on a satisfactory pitched or consolidated gravel foundation was successful for all roads except city streets. Claims were made by companies engaged in the sale of bitumen that very light veneers of bitumen laid upon the surface of well-drained earth roads were capable of carrying large volumes of traffic, but it was very doubtful whether those surfaces could be kept in order without heavy maintenance costs. The section of the road shown in *Fig. 2* (p. 206 §) was typical of those constructed by machine. It was found that in many cases the borrow-pit silted up owing to erosion from the shoulders, or in other cases eroded if the longitudinal section of the road were steep. Small traverse channels at short intervals to carry the water clear of the road formation were essential. The modern elevating graders were a great improvement on the old horse-drawn elevating graders used 30 or 40 years ago to make a similar formation for railways in flat country. Some further particulars of the rooter capable of handling boulders 1 metre in diameter would be welcomed.

The quantities of bitumen employed per square yard appeared to be about twice those employed elsewhere for surfaces carrying at least 300 vehicles per day; but in the present case, where bitumen was possibly very cheap, there might be some reason for the much heavier treatment. The value of the Paper would be greatly enhanced if particulars could be given of the cost of the bitumen and the surfacing of the road.

§ *Ibid.*

**Mr. S. W. F. Morum** observed that it was doubtful if engineers wh had not had experience of Near East desert-conditions would realize wha was involved by the difficulties of climate, lack of water and supplies—no only of food but also of suitable aggregate—and, furthermore, the difficult of successful operation of mechanical plant and transport vehicles handle by native personnel. Mr. Morum's experience in Iran with America road-making equipment while with the Anglo-Iranian Oil Company, Ltd led him to agree with the Author that, despite cheap labour, mechanizatio not only simplified the handling of the job from the executive point of view but also effected very considerable savings; in fact, in Iran, the cost o formation-work for roads could, by the use of "carry-all" scrapers, b reduced to about one-tenth of the hand-labour costs. (The actual figur was naturally effected by the low cost of fuel for the oil company's ow uses and by the company's facilities for importing plant duty-free.)

For work in Iran where roads were constructed with a 24-foot widt at the crown, elevating graders would not have enabled the spoil-pits t be situated sufficiently far from the toe of the bank, so carry-all scraper were employed. Machines of 6 cubic yards capacity were used, since suc machines gave quicker turn-round times than the larger ones. Th machines were worked across the bank instead of in the usual manne with the result that the travelling time was reduced to the minimum and i was possible to obtain an average output of some 2,000 cubic feet per hou for a 24-hour day. Despite the fact that all the drivers were Persians an were trained on the job, the machines proved to be exceptionally reliable.

Generally, experience in Iran showed that it was almost impossibl to limit axle-loadings on vehicles owned by the native contractors, an nominal 12-ton four-wheel lorries were often found loaded with fron 18 to 20 tons; in one case a lorry was found to be working with a wheel load of 12 tons on its rear wheels. The Author admitted that on a deser road speed-control was not possible, and it would be of interest to kno what methods he had arranged to stop the road from being abused b over-loading.

Had the typical road-section shown in *Fig. 2* (p. 206 §) proved success ful? In Iran the protective banks shown on the outside of the borrow ditches in *Fig. 2* had been abandoned, as they had proved to be ineffective being easily damaged by animals and floods. Also, as the bank wa presumably made in the clay and far below optimum moisture-content what percentage of settlement was expected? Experience in Iran wa that a settlement of the order of 25 per cent. had to be allowed for.

With regard to the culverts and causeways constructed east of th lava belt, what percentage drainage-opening per mile did those giv in the shallow flows, and was allowance made for wind-driven floods That point, which entered into several of the Iranian projects—whic

§ *Ibid.*

had been suspended owing to the war—was the cause of some controversy, and Mr. Morum was of the opinion that for wind-driven floods alone, openings of the order of 100 square feet per mile were required.

With regard to the carpet, had the Author considered the methods of soil-stabilization that were being employed in the United States, particularly in the arid western states? Mr. Morum would have thought that those methods could have been used in some places with considerable economy. Presumably the bitumen referred to as "80 per cent." was the "Shelspra" F.80 grade. What considerations had led to the employment of that grade, and how was it standing up in what was bound to be a relatively flexible road-mat, under winter temperature conditions? The "Shelspra" F grade contained a 30/40-penetration base, whereas the majority of the United States specifications for similar work called for a base with a mean penetration of 80/100; a base harder than 60/70 penetration would never be used. Mr. Morum's experience in Iran led him to conclude that the M.C. grade (80/100-penetration base) gave excellent results for flexible carpets; and that a 40/50-penetration bitumen, even in the very high Iranian temperatures (sun maximum 187·5° F.), using the Iranian product, which was comparatively susceptible to temperature, gave the best results for hot mixing for asphalt carpets, provided that the filler-content was correctly handled.

What was the result of the contact of crude oil and bitumen in the road carpets? Experience with Iranian fuel-oil in contact with bitumen was very unfortunate. Roads originally made by dressing with fuel-oils, which proved unsatisfactory, were very difficult to re-treat with bitumen. The general practice in Iran was to run a fuel-oil carpet to destruction before attempting repairs more extensive than the normal pot-hole filling.

What had led the Author to prefer a portable-machine mix for the lava-belt section, rather than either a grader-mix on the site, or a plant-mix delivered on to the site from mixing plants stationed at the quarries or some other central point?

**The Author,** in reply to the Correspondence, observed that he appreciated Mr. Hamilton's regrets that the road was not built to the highest specification of load-carrying capacity. However, the British Government were not prepared to spend a greater sum and the local governments did not assist financially, so that the best had to be done with the money allocated.

The Haifa–Baisan–Jordan route was a better alignment in every way than the Nazareth–Tyberias route and it was hoped, when financial considerations would permit, that the Palestine Government would bring the latter route up to the standard required. There would still remain the Wadi el Arab section from Jisr el Majami to Irbid, where the road was cut into the precipitous sides of the *wadi* for many miles, with hairpin bends. That had already been considerably widened, but if it was to be made to take such vehicles as the Marmon–Herrington vehicle-train, the

necessary widening and re-aligning would cost a large sum and might enta a new alignment for much of the distance.

The Author could not agree altogether with Mr. Hamilton's statemen that section 2 was, on the whole, a fair natural road, capable of carryin (for about 50 weeks in the year) the heaviest classes of vehicle. Portion of that section, including most of the stretch from H.4 to H.3, were im passable even to light cars after rain, and frequent stoppages of traffic— often for a week at a time—occurred each winter.

The construction of the route within Iraq was a matter for the Ira government, and the Author did not know why work at that end was no proceeding. It was expected that Iraq would work concurrently on tha portion where it was not already an all-weather route. It was understoo that an irrigation scheme was about to be put into effect to utilize the flood water from Habbiniyah lake and to prevent its overflow in the wet season which caused a wash-out of a considerable length of road each season. I was understood that the new portion of road from Ramadi toward Rutbah had failed through traffic and weather, owing to the poor specifi cation of the surface mat.

" Crown Agent for the Colonies " standard heavy-bridges were erecte which carried a train of vehicles consisting of a tractor and a trailer with the following axle loads :

| | |
|---|---|
| Tractor, front axle . . . . . | 4 tons. |
| ,, rear axle . . . . | 8 tons. |
| Trailer . . . . . . . . | 4 tons on each axle. |

The control of the class of vehicle using the road was not difficult as there were no inhabitants except at the ends of the road, where efficient custom controls existed. The road was controlled at intervals by the Arab Legion and nothing moved over all that expanse of desert without their knowledge Examination at the ends could ensure no overloading, as there were no extra loads to be picked up *en route* in that barren desert.

Mr. McConnel had stated that a heavy and costly bitumen surface had been adopted compared with the practice in South Africa and Australia where 1½ to 2 inches on a satisfactorily pitched and consolidated gravel foundation was successful for all roads except city streets. When a satis factory foundation was constructed on the Haifa–Baghdad road, as in the Irbid–lava belt length, a 2-inch consolidated premix was laid. On the desert sections of that road east of the lava belt the formation was of clay. No gravel or other material was available to strengthen it. Stone was available at a reasonable cost of extraction in limited quantities only, and it had been found considerably cheaper to use a thicker mat of bitumen macadam (4 inches in that case) on the clay-formation than to use a thinner bitumen mat and about double the quantity of stone to strengthen the formation. It was also essential that a thoroughly waterproof mat, that would remain waterproof under considerable wear, should be used over a clay-formation, owing to the risk of water getting under the surface and

causing collapse. That specification was found to be the minimum from results on similar formations at the Habbiniyah R.A.F. camp.

On those portions of the pipe-line road that were utilized in the lava belt, a thick mat was laid in order to level out inequalities in the old soling as that was cheaper than picking-up and relaying it. The Author did not know full details of the cost, but a fairly accurate average cost for the various sections was £400 per kilometre, using a 10-centimetre thickness of bitumen macadam mixture, excluding sealing-cost. Bitumen cost approximately £7 per ton at the railhead.

A portable machine-mix was originally selected for the lava section rather than a grader-mix, as it was considered, on the advice of the representative of the manufacturer of the grader machinery, that on the rough soling a blade could not pick up the mix satisfactorily. That was overcome in practice, as described on p. 211 §. A plant mix was not used, as the stone in the lava belt lay to hand, in boulders of suitable size for crushing, all along the route, and transport was saved by the use of the tractor-drawn and operated crushers which crushed the stone and laid it in windrows along the route.

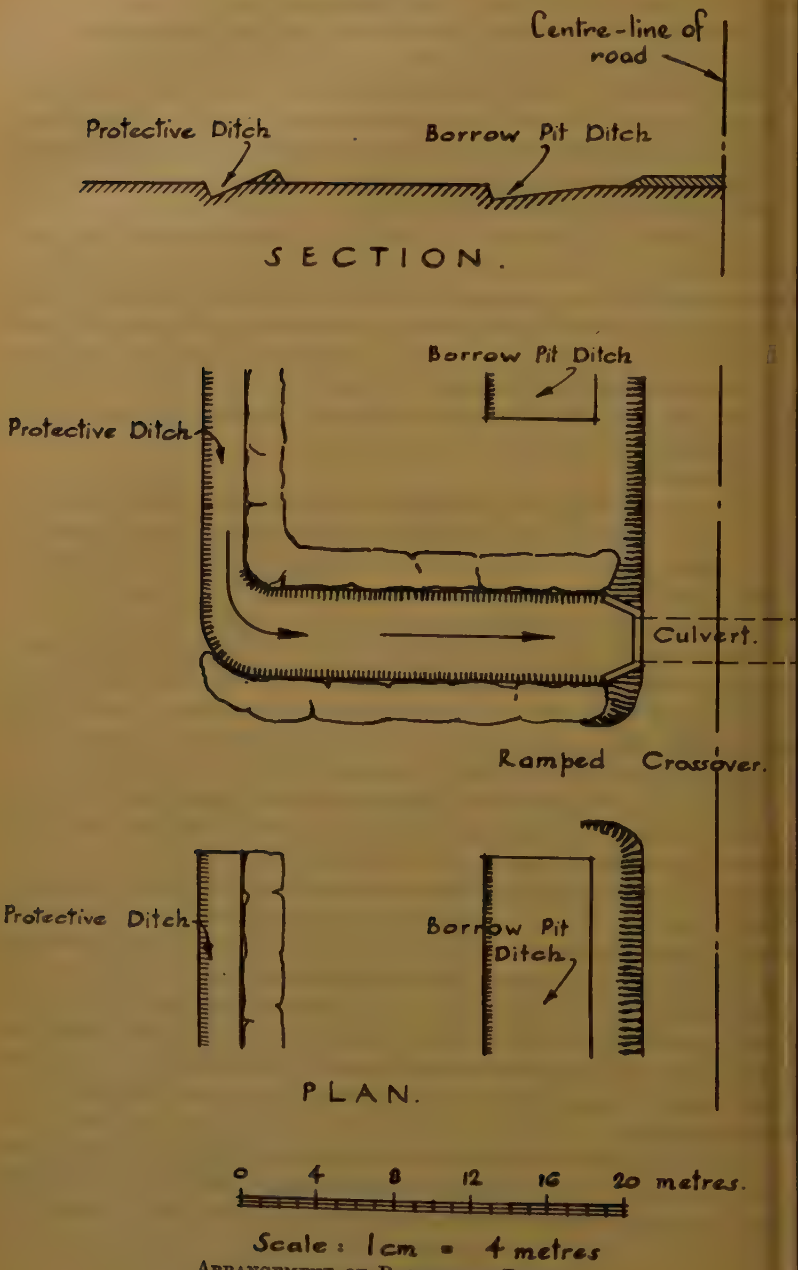
The cost of raising the road formation by elevating grader, compacting it in layers by blade-grader, levelling to grade as necessary by a 12-cubic-yard scraper, and finishing with a crown, was 2 *piastres*, say, $4\frac{3}{4}d$ per cubic metre. That did not include the cost of fuel-oil, which was negligible since it was taken from the pipe-line. The Author had no costs of cut and fill by 12-cubic-yard scrapers alone. He wondered if Mr. de Leeuw's costs were due to using a number of small "Tumblebug" 2-cubic-yard scrapers each with a D.4 tractor and European driver, as were often used in Palestine, as he considered that the use of a 12-cubic-yard carry-all scraper and D.8 tractor would, in comparison, make a great saving.

Mr. Morum had mentioned soil-stabilization: presumably he referred to the practice of mixing the top 3 or 4 inches of the formation with a road-oil, bitumen emulsion, or cut-back bitumen to increase its load-bearing capacity. In that case the formation was almost entirely of clay, with a small proportion of stone brought up by the excavators in large lumps and in very variable quantity. There was no sand or gravel available in the soil or nearby. The soil was, therefore, entirely unsuitable for bituminous treatment. On the other hand, very fair-quality limestone was available locally and that presented a means of producing a cheap and high-quality surface requiring little maintenance which was homogeneous and of sufficient thickness to remain waterproof under wear, and thus protect the clay-formation.

The bitumen at first used in the mix in situ was of a special grade made from 40/50-penetration bitumen; but that was found to be too heavy. The Author was informed that, subsequently, all mix-in-situ work was done

§ *Ibid.*

*Fig. 5.*




ARRANGEMENT OF PROTECTIVE DITCH.

on lines similar to those of the second experiment referred to on p. 211 §. In that process of admixture of cut-back and crude oil, the heavy ends of the latter acted as a flux for the basic bitumen of the cut-back, and the residual film around the stone particles gave at least as flexible a carpet as cut-back made from 80/100-penetration bitumen. The Author understood that it did not necessarily follow that a cut-back made from 30/40-penetration bitumen would give a harder residual film than one made from 80/100-penetration bitumen. By using a heavier, that was, less volatile solvent with the 30/40-penetration bitumen a residual film as soft or softer could be obtained as from 80/100-penetration bitumen with a lighter solvent. The Author could not speak with personal knowledge about the mixture of crude oil and cut-back bitumen, but he had been told that the process described in the last paragraph on p. 211 § had been adopted with success.

With regard to the borrow-ditch, it was found that that silted up in the clay section, owing to erosion from the shoulders where the longitudinal section was steep. That was caused by the entry of water from the surrounding desert. Steep sections were, however, not common. To prevent water flowing into the ditch, a protective ditch was dug by a blade grader, and the water flowing in that ditch was carried under the road through culverts to the down-hill side of the road at frequent intervals, and was led off in transverse channels away from the road (*Fig. 5*). The Author did not remain long enough on the work to see what maintenance would be required, but it was always anticipated that the protective ditches and earth berms of the road would require blade-maintenance each year by auto-patrol grader. Camels did a certain amount of damage to the ditches during migration of the tribes; but that was not very serious, as they preferred to use the crossing-places provided at the top of each grade rather than to face the steep borrow-ditch.

Settlement in the bank was less than 10 per cent., probably owing to each 6-inch layer of soil being consolidated by a blade-grader and D.8 tractor, and being finished by a 12-cubic-yard scraper. The consolidating effect of those heavy machines was exceedingly good.

The rooters used to move boulders of up to 1 metre diameter were "Le Tourneau" heavy rooters drawn by D.8 caterpillar-tractors. Those rooters were provided with three prongs, about 3 feet long. In moving large rocks it was found to be more effective to use one prong only.

Since the war the Author had heard little news of the road, except that it had stood up to its second rainy season well, so he could only presume that the road section was, generally, a success.

---

§ *Ibid.*

Paper No. 5205.

# "The Hydraulic Problem Concerning the Design of Sewage-Storage Tanks and Sea-Outfalls." †

By John Rupert Daymond, M.Sc., Assoc. M. Inst. C.E.

## Correspondence.

**Mr. L. B. Escritt** observed that the method devised by the Author was an ingenious and effective treatment of the simplest case of an elusive problem. As the Author stated, "The problem reduces to a differential equation which cannot be solved explicitly, whilst an algebraic solution is too involved to be of practical use."

The time taken to empty a tank with vertical sides or other regular proportions might be found by a simple calculus equation when the discharge was through a pipe, orifice, or combination of pipes, etc. When a constant flow into the tank had to be considered the time might be calculated by a more elaborate, but still reasonable, calculus equation. It was the effect of the sine-curve motion of the tide which made the problem difficult. The Author had solved the problem satisfactorily in the case where a tank with vertical sides discharged into the sea.

However, in most cases the problem was more complex. In the first place, the vertical-sided tank was not common. The tank might be in the form of a cylindrical inclined sewer, or of an inclined tank sewer with straight sides and arched crown and invert. The problem had sometimes to be related to the discharge of natural basins of irregular form, such as, for example, the discharge of a river through a sluice-gate. There was also the case of discharge on those parts of the coast where the tide was double and did not follow the sine-curve.

For the foregoing reasons, rule-of-thumb methods had been devised individually by several investigators for finding the discharge of a tank under all sorts of conditions. They were all variations of the same principle and were the natural conclusions reached in attempting to find a reasonable solution to a problem which could not be approached mathematically. Mr. Escritt's own method, which he had entitled the "Graph Method," was carried out in the following way:

1. The highest and lowest spring tides were marked out, and, between those limits of amplitude, a sine-curve with a period of 12 hours was

† Journal Inst. C.E., vol. 13 (1939–40), p. 217 (January 1940).

plotted with levels as ordinates and times as abscissæ. That represented tidal conditions for most places. Where the tide did not rise and fall as a sine-curve, actual levels had to be taken and the curve plotted.

2. To the same vertical scale, but to any horizontal scale, a curve, giving the surface-area of sewage in the tank at any sewage-level, was drawn.

3. The centre-line of the outfall pipe at the lower end was drawn.

4. Starting with the tank almost empty, and the rising tide just level with the centre-line of the outfall-pipe, the head from tide-level to sewage-level was measured.

5. The discharge, in cubic feet per 5-minute period, was calculated, and from it was deducted the flow entering the tank.

6. The surface-area of the tank at sewage-level was measured; the difference of inflow and outflow divided by that area gave the rise (or fall) of sewage-level in the tank.

7. The new level after the 5 minutes' interval was marked.

8. The new head between the new tide-level and the new sewage-level was measured, and the operation was repeated until the tank had filled and emptied.

Such a process gave a curve showing the level of sewage in the tank at any time, and the time necessary to empty the tank on a falling tide, for a particular size of outfall-pipe.

The size of outfall-pipe was first determined by assuming the average head between the water-level in the tank and sea-level, and it was then checked, to see if it could discharge the contents of the tank in the required time.

In few cases had allowance been made for the fact that sea-water had a higher specific gravity than sewage, and that, in consequence, when the level in the tank and the level in the sea were the same, there would be, in the absence of any tide-flap, flow from the sea into the tank. Allowances for differences of specific gravity could be made in cases that required such a degree of accuracy.

**Mr. David Lloyd** wished to know whether the problem did not really consist of designing a tank of such a capacity or depth, and an outfall of such a diameter, that the tank was filled and emptied over a tide.

The solution appeared to have further applications, giving, for example, the rise and fall in a tank of depth greater than the critical depth $H-D$. In that direction, the solution was of interest to water engineers. Town supplies were generally provided from a constant rising main into a storage reservoir. The draw-off was subject to a diurnal fluctuation, creating a resisting head very similar to the tidal head. The corresponding "$z$-curve" (which was generally superimposed on a break-down storage) was becoming of greater economic importance. Its derivation had been looked on largely as one of trial and error.

Mr. Lloyd was doubtful of the validity of equation (2) (p. 220 §), and the Author had to some extent anticipated Mr. Lloyd's view by his remarks on p. 229 §. In the prototype the outfall was submerged, and, furthermore, the fluid was rather different from water. It appeared to Mr. Lloyd that the power $n$ in $q=kh^n$ might not be 0·5, although near to it. Would a 20-per-cent. difference in the value of $n$ alter the transformation-ratios to any appreciable extent?

**Mr. B. T. Rees** admired the ingenious method by which the Author had approached the problem, but proposed to discuss the way in which the results worked out in practice. Investigation of example 1 (p. 231 §) by previously-known methods, assuming $A=7500$, gave $\varDelta=5{\cdot}80$, as compared with the Author's $\varDelta=5{\cdot}86$; it would be seen also that a pipe 1·5 foot in diameter (as compared with the Author's $d=1{\cdot}566$ foot) would discharge the tank as shown by c′a′b′ in *Fig. 12*. It should be remembered, however, that there were an infinite number of pipe-sizes larger than 1·5 foot that would also discharge the tank, and so would certain smaller sizes of pipe; the Author's figure of 1·566 foot could not, therefore, be rigidly adhered to. *Fig. 12* showed a long period of free discharge. The length of that period was usually incorporated in the design so that the tank might be entered and cleaned. In certain cases, storage tanks were built in parallel, and so any one of them could be isolated for cleaning purposes; in such cases no period of free discharge was necessary, and pipes of much smaller diameter than 1·566 foot would be sufficient.

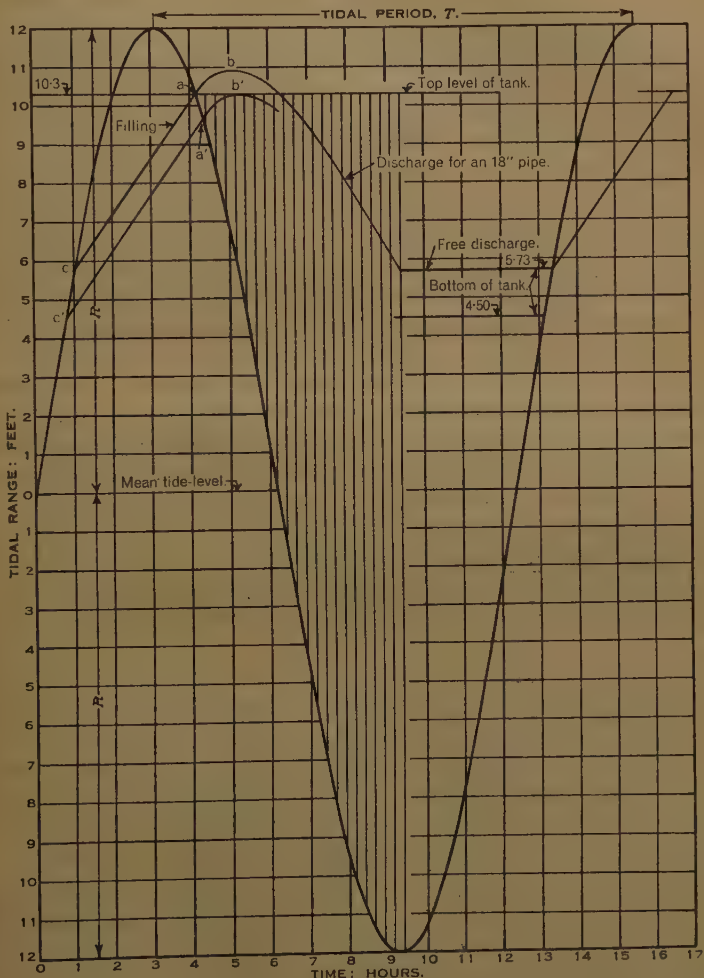
In example 2 (p. 232 §) the Author had assumed a commercial size of pipe (1·5 foot diameter), other data being similar to those of example 1, and had set out to prove that, if the pipe-diameter were changed from 1·566 foot to 1·500 foot, then the area of the tank would have to be increased from 7,500 square feet to 12,640 square feet, whilst the depth of the tank could be reduced from 5·86 feet to 2·92 feet. There appeared to be a lack of balance in such results which the Author might be able to trace. For instance, what would happen should incrustation or other causes slightly reduce the diameter of the pipe?

Mr. Rees suggested that the lack of balance in the results might be due to the fact that the Author had tried to make one problem of a design which really involved two distinct problems: firstly, storage, and secondly, discharge. Those two problems could be solved separately by means of the diagram shown in *Fig. 12*. At some point c, the rising tide closed the flap and storage took place uniformly until a point a was reached on the tide-curve when discharge should become noticeable. If the outfall pipe were of considerable size, then the discharge-curve would follow closely the tide-curve. Since, however, the pipe was relatively small, the discharge curve showed a further rise in the tank-level until there was sufficient head to produce a quick discharge. The point a was on the top-water

§ Page numbers so marked refer to the Paper (Journal Inst. C.E., vol. 13 (1939-40), p. 217 (January 1940).—SEC. INST. C.E.

design-level (10·3 feet) of the tank, and for the particular tank considered that level should not be exceeded. Any number of lines, ac, might be drawn from a; such lines were the filling lines whose gradients depended

*Fig. 12.*




on the areas of the tanks considered, since the inflow was assumed to be constant. Table II was a comparative list of such tanks. The storage-depths were read directly after plotting the filling gradients. Total storage-time was found in the same way.

The ultimate factor in tank design was the question of cost, which

could be roughly estimated from the last column of Table II; the Table should be used only as a preliminary guide.

It would be noted, in *Fig. 12,* that the discharge for an 18-inch-diameter

TABLE II.

| Area of tank: square feet. | Inflow into tank: cubic feet per minute. | Rise in tank-level: feet per hour. | Storage-time: minutes. | Storage-depth: feet. | Tank-capacity: cubic feet. | Comparative storage-tank dimensions: feet × feet × feet. |
|---|---|---|---|---|---|---|
| 5,000 | 180 | 2·16 | 230 | 8·30 | 41,500 | 8·30 × 8·30 × 620 |
| 6,000 | 180 | 1·80 | 208 | 6·24 | 37,440 | 6·24 × 6·24 × 962 |
| 7,000 | 180 | 1·543 | 195 | 5·03 | 35,210 | 5·03 × 5·03 × 1392 |
| 7,500 | 180 | 1·44 | 189 | 4·57 | 34,275 | 4·57 × 4·57 × 1643 |
| 8,000 | 180 | 1·35 | 185 | 4·19 | 33,520 | 4·19 × 4·19 × 1909 |
| 9,000 | 180 | 1·20 | 177 | 3·56 | 32,040 | 3·56 × 3·56 × 2528 |
| 10,000 | 180 | 1·08 | 172 | 3·10 | 31,000 | 3·10 × 3·10 × 3225 |
| 11,000 | 180 | 0·982 | 168 | 2·75 | 30,250 | 2·75 × 2·75 × 4000 |
| 12,000 | 180 | 0·90 | 164 | 2·46 | 29,520 | 2·46 × 2·46 × 4880 |

pipe took place while further storage was also occurring; and, since the top design-level of the tank was fixed at 10·3 feet above mean-tide level, the discharge curve had to be lowered by shifting the curve ab to the position a′b′, where it was tangential to the 10·3 foot level. It would be found that there was scarcely any inaccuracy in doing that, and the

TABLE III.—DISCHARGE CALCULATIONS FOR AN 18-INCH-DIAMETER PIPE.

| Period of time: minutes. | Storage: cubic feet. | Head: feet. | Hydraulic gradient. | Outflow: cubic feet per minute. | Change in storage: cubic feet. | Final level in tank: feet. |
|---|---|---|---|---|---|---|
| 0–15 | 34,275 | Nil. | — | — | + 2,700 | 10·67 |
| 15–30 | 36,975 | 1·17 | 1/4700 | 99 | 15 (180–99) = + 1,215 | 10·82 |
| 30–45 | 38,190 | 2·44 | 1/2255 | 144 | 15 (180–144) = + 540 | 10·90 |
| 45–60 | 38,730 | 3·61 | 1/1523 | 175 | 15 (180–175) = + 90 | 10·90 |
| 60–75 | 38,820 | 4·90 | 1/1122 | 204 | 15 (180–204) = – 360 | 10·84 |
| 75–90 | 38,440 | 6·24 | 1/882 | 230 | 15 (180–230) = – 750 | 10·76 |
| 90–105 | 37,690 | 7·56 | 1/727 | 254 | 15 (180–254) = – 1,110 | 10·60 |
| 105–120 | 36,580 | 8·90 | 1/618 | 275 | 15 (180–275) = – 1,425 | 10·42 |
| 120–135 | 35,155 | 10·27 | 1/536 | 296 | 15 (180–296) = – 1,740 | 10·20 |
| 135–150 | 33,415 | 11·50 | 1/478 | 313 | 15 (180–313) = – 1,995 | 9·92 |
| 150–165 | 31,420 | 12·72 | 1/433 | 329 | 15 (180–329) = – 2,235 | 9·64 |
| 165–180 | 29,185 | 13·84 | 1/397 | 343 | 15 (180–343) = – 2,445 | 9·30 |
| 180–195 | 26,740 | 14·80 | 1/372 | 354 | 15 (180–354) = – 2,610 | 8·97 |
| 195–210 | 24,130 | 15·87 | 1/347 | 367 | 15 (180–367) = – 2,805 | 8·60 |
| 210–225 | 21,325 | 16·60 | 1/332 | 375 | 15 (180–375) = – 2,925 | 8·15 |
| 225–240 | 18,400 | 17·20 | 1/319 | 383 | 15 (180–383) = – 3,045 | 7·75 |
| 240–255 | 15,355 | 17·75 | 1/310 | 388 | 15 (180–388) = – 3,120 | 7·35 |
| 255–270 | 12,235 | 18·10 | 1/304 | 390 | 15 (180–390) = – 3,150 | 6·94 |
| 270–285 | 9,085 | 18·29 | 1/301 | 394 | 15 (180–394) = – 3,210 | 6·50 |
| 285–300 | 5,875 | 18·25 | 1/302 | 393 | 15 (180–393) = – 3,195 | 6·12 |
| 300–315 | 2,680 | 18·07 | 1/304 | 392 | 15 (180–392) = – 3,180 | 5·73 |
| 315–330 | Nil. | Period of free discharge has begun. | | | | |

line a′c′, parallel to ac, was the new filling gradient, the depth of storage then being 5·80 feet instead of the original 4·57 feet.

For simplicity, the discharge for an 18-inch-diameter pipe only was plotted in *Fig. 12*, but any number of outfall-pipe sizes could be treated in the same way, and a selection made of the pipe-sizes most favourable for the discharge conditions.

Table III showed how the discharge of the 18-inch pipe was calculated for the original condition. The period of discharge was shown on the diagram, and was divided up into short periods of 15 minutes, commencing at point a, and the calculations were made accordingly. Table III then became a tabulation of inflows and outflows according to the variable heads, which were read off *Fig. 12* directly with reference to the tank-storage level from Table III.

**The Author,** in reply, wished to point out that any variation from $A$ constant precluded the possibility of a general solution to the problem along the lines given in the Paper. An appeal to experiment would produce restricted results, because transformation from the model to nature was not possible when $A$ was a function of $z$. Hence, for $A$ variable, a satisfactory solution might ultimately be obtained after a tedious process of trial and error, by deriving a series of particular solutions using an approximate "step-by-step" process, as suggested by Mr. Escritt, and presumably used by Mr. Rees in deriving Table III. For that purpose the necessary modification of the graphical method of the Paper would be evident. It was also evident that such methods yielded no immediate information on the required size of tank and outfall: they merely indicated, by the derivation of a $z$-curve, whether or not assumed sizes of tank and outfall were satisfactory; there still remained the question as to the most economical scheme.

It was gratifying to find that the results of example 1 of the Paper agreed very closely with those obtained by Mr. Rees. There was, however, no "lack of balance" in the results of examples 1 and 2. They were given merely to illustrate the use of "the field," and it was not suggested that any result should be rigidly adhered to. On the contrary, it was stated very clearly that there were, in most cases, a large number of possible solutions (see p. 234 §). Mr. Lloyd would also find there (p. 234 §), and in various other parts of the Paper, the reply to his question on the nature of the problem and its solution.

The apparent "lack of balance," referred to by Mr. Rees, prompted him to conclude that two distinct problems were involved, one of storage and the other of discharge. It could not be too strongly emphasized that that conclusion was entirely erroneous. It was clearly shown in the theoretical discussion, and equally clearly demonstrated by the "field" results, that $m$ and $s$, the respective tank and outfall factors, were closely related. Despite his remarks to the contrary Mr. Rees tacitly accepted

§ *Ibid.*

that relationship in his discussion of *Fig. 12* (p. 519, *ante*), where, in addition, there was much reiteration of what was given in the Paper.

It was not very evident where Mr. Rees's approach to the problem displaced the more direct and convenient method of the Paper. From a study of *Fig. 12*, in addition to the unnecessary approximation involved in moving the $z$ filling curve bodily, it was suspected that most of the conclusions arrived at were invalidated by assuming that the $z$-curve for free discharge cut the tide-curve horizontally. That was impossible unless $m$ was zero (that was to say, unless $A$ was very large or $Q$ very small). Similarly, there was an error in assuming a depth of 5·80 feet, because storage would begin before the point c′ was reached. A design yielding a lengthy period of free discharge was not economical, since it involved a bigger outfall than was really necessary. It should be remembered, too, that the longer the period of discharge from the tank itself, the greater was the dilution of the sewage in the tidal water, an advantage to be utilized when discharge was permissible at all stages of the tide. If a period of free discharge were aimed at for tank-cleaning or as a margin of safety, that might be done by increasing $d$ over the "field" result, as mentioned in p. 234 §.

It was not possible to say precisely what effect a 20-per-cent. increase in $n$ would have on the results. If $n$ in nature were greater than 0·5, then all "field" results were on the safe side. Mr. Lloyd was entitled to doubt the validity of equation (2); at the same time it was agreed that $n$ was always near to 0·5, and that was the value obtained for the model. Therefore, bearing in mind the approximate nature of hydraulic laws, the "field" results should be acceptable. It might be mentioned, in reply to the point raised by Mr. Rees, that there was freedom in the choice of $k$, and its value should be such as to cater for possible incrustation of the outfall in the future.

The problem of the Paper did bear some relation to that of the design of service reservoirs. The results, however, could not be extended to solve those water-supply problems.

Referring to Mr. Escritt's discussion on the "Graph Method," it should be borne in mind that a solution satisfactory for spring tides might not necessarily be satisfactory for other tides, as noted in p. 232 §. Hence the tide considered by Mr. Escritt might not be the critical one. It was not clear what was meant by the statement that "The size of outfall-pipe was first determined by assuming the average head between the water-level in the tank and sea-level. . . ." If, as was presumed it was meant to be, the head were measured between the top permissible tank-level and mean tide-level, that head was $H$. Hence, using the symbols of the Paper, $k$ was immediately obtained from $Q=kH^{\frac{1}{2}}$, since $Q$ and $H$ were both known, whilst $d$ was ultimately derived from equation (13$b$) (p. 232 §).

§ *Ibid.*

If, for the same $m$, $d_1$ were the required diameter as given by "the field" and $d$ were as defined above, it would be easily seen that

$$s_0 k_1 = k/H_0,$$

$k_1$ being the constant for $d_1$. Then, by tabulating values of $s_0$ and $H_0$ as derived from "the field" for $m_0$ constant ($Q$ and $A$ constant), it would be found that

$$k_1 > k,$$

and hence that $$d_1 > d.$$

Therefore, the diameter as assumed would not be large enough to discharge the contents of the tank before it filled up again.

It would also be found, by similar reasoning, that in practically all cases $d_1 > d$ if the head were now measured between the mean level in the tank and the mean tide-level. In that case the head was $(H+D)/2$, and the equation to be considered in tabulating values of $H_0$, $D_0$, and $s_0$ from the field for $m_0$ constant, was

$$s_0 k_1 = k\left(\frac{H_0+D_0}{2}\right)^{\frac{1}{2}}$$

It would be noted that $H_0+D_0$ had to be greater than zero.

Compared with other factors which had to be considered in practice (effect of on-shore wind, changes in barometric pressure, etc.), the question of the relative density of sewage and sea-water was of minor importance. If necessary, an allowance could be made by adjusting tide-levels in relation to the level of the sewage in the tank.

There were certain errors in the Paper which only revealed themselves on a careful reading, and therefore should not have prevented a clear understanding of the context:

p. 224§, first part of equation (12), *for* $m_0 = m\frac{I}{R}$, *read* $m_0 = m\frac{T}{R}$,

p. 232§, equation (13*b*), *for* $k = \pi\left(\frac{gd^5}{32fl}\right)$ *read* $k = \pi\left(\frac{gd^5}{32fL}\right)^{\frac{1}{2}}$

p. 233§, second line following Table I, *for* $m = 7{\cdot}25\ 10$, *read* $m = 7{\cdot}25 \times 10^{-4}$

p. 236§, last paragraph, line 3, *for* $h_5$—line *read* $h_s$—line

§ *Ibid.*

Paper No. 5220.

## "The Kidlington Bridges." †

By Isaac Kursbatt, B.Sc., Assoc. M. Inst. C.E.

### Correspondence.

**Mr. M. F. Barbey** observed that probably the most obvious schem would have been to continue the old line of main road southwards acros the end of Langford lane by demolishing one or more of the cottages. Th diversion would then have run behind the Railway hotel and have swep eastwards to cross the railway, probably in a two-span bridge abou 100 feet south of the present bridge. Langford lane need not then hav been diverted, the canal would have been crossed at an angle of 62 degree by a new bridge just clear of, and to the south of, the old one, and the cana diversion would also have been unnecessary. The old canal bridge coul have been retained to serve the hotel and the railway goods-yard. Th approach to the latter would have passed under the new road by mean of a bridge built with the comparatively new superstructure from th existing railway bridge, half at a time. The new road would have con tained reverse curves of 1,000 feet radius, instead of 3,000 feet radiu as in the adopted scheme, but the gradients should have been simila Other disadvantages, however, would have been the possibility of havin to rebuild the hotel, and some additional complications at the railway The adopted scheme, therefore, with its greatly improved curves, was n doubt amply justified.

Were the "old brick retaining walls" (p. 247 §) strutted at the top b an arch, or were they free? What foundations and drainage had bee provided, what exactly was the type of fill, and were the weep-pipes stil functioning?

Presumably smoke-plates were not provided at the railway bridge o account of restricted headroom. Was it not thought worth while t form smoke-shields on the two faces of the bridge, with small concret cantilever slabs, to prevent the disfigurement apparent in *Fig.* 7 (facin p. 248 §)? How was the bitumen paint applied, and how thick was it Was it necessary to heat it, or to thin it before application? How wa it standing up to engine-blast?

---

† Journal Inst. C.E., vol. 13 (1939–40), p. 237 (January 1940).

§ Page numbers so marked refer to the Paper (Footnote (†) above).—Sec. Ins C.E.

Were the piles that were driven practically to refusal in waterlogged sand (p. 248 §) redriven later comparatively easily? Had any resistance–penetration curves been drawn for them or for the concrete sheet-piles?

It was mentioned on p. 249 § that variations in the span–length occurred with the welded portal frames. Was that due to distortion during welding? What were the welding procedure, the method of holding the parts, and the sequence of runs?

**The Author,** in reply, observed that the suggested alternative scheme put forward was of interest, but the greatly improved curves of the scheme as carried out had decided advantages. Mr. Barbey had, however, overlooked the clearances necessary for the bridge over the railway goods-yard approach, as well as extensive alterations to the existing station and platform. In addition to the demolition of the canal-side cottages, it would have been necessary to remove the stationmaster's cottage on the southern approach to the railway bridge. Last, but not least, the Railway Company's agreement was essential, and it was doubtful whether they would have agreed to that diversion when a more suitable crossing was possible from their point of view.

The retaining walls in the old structure were built-in monolithically with the abutment and arch as described (p. 247 §). In that sense, they might be considered strutted about one-third of their height from the top. The factor contributing most to the stability of those walls was probably the dry condition of the back-fill. It consisted of gravel with a slight admixture of clay. During the excavations for the new abutments no shoring was used for the full depth of 20 feet. It was slightly battered on the face, which was protected by tarpaulins. Incidentally, a Ruston-Bucyrus navvy operated as a crane was at work on top. It appeared that the thickness of retaining wall demanded by the rigid considerations of either Rankine's or Coulomb's theory was excessive, provided that the surface of the fill was kept watertight. The later theories of Professor K. von Terzaghi seemed to be justified by that experience. There were no weep-holes in either the abutments or the wing-walls.

"Callendure" bitumen paint was made from "Gilsonite" bitumen with suitable fluxing agents and driers. It required neither heating nor thinning before application. In fact, thinning was apt to destroy the efficacy of the paint. It was applied in two separate coats by means of a spray-gun from a pressure-tank. Generally, it seemed to be standing up very well. Only at several bays where the continuous hot blast due to waiting goods trains had been extraordinarily severe had there been any signs of peeling. The thickness of the two coats was, of course, minute, and was not readily measurable.

A single steel test-pile had been driven outside the north cofferdam of the canal bridge to a depth of 23 feet below water-level after the first blow-out

§ *Ibid.*

occurred. That was done to find out whether rock existed at a reasonable level below the sand. On withdrawing that pile, blue clay was found in the interlock of the lower 1 foot 6 inches. The conclusion drawn from that test was that the sand caused binding in the locks of the main steel coffer-dam, giving the impression of rock bottom. After the blow-out, the piles were re-driven comparatively easily, since the water had washed out the binding sand. The concrete sheet-piles driven inside the cofferdam were used chiefly to tighten up the sand foundation in the enclosure. Attempts were made to obtain resistance–penetration curves with the first few piles, but it was found that each pile behaved differently. The last piles drove barely 3 feet. It was possible that the closed box of steel piling filled with concrete would have been satisfactory without the addition of the concrete piles. The latter were driven only as a precautionary measure. To have recorded the penetration of each individual steel pile for a specific number of hammer-blows would have been both tedious and lengthy, and would have given little useful information.

The following was the welding procedure of the canal-bridge portal-frame girders: (*a*) butt-welds were made to the web; (*b*) the stiffeners were welded to the web; (*c*) the flange-plates were welded to the web working from the leg bases upward to the crown from either side; (*d*) finally the plates over the crown were welded. In each case, suitable clamps were used for the component parts, and tack-welds were applied at convenient points. Specially-designed clamps were used for the welding of the flange-plates to the web. All those operations were carried out with the girder lying on its side. The Author was not able to give the details of the welding sequence, since the work was done a long way from the site. Practical details were left to the discretion of the Fairfield Shipbuilding and Engineering Company, Ltd. Examination of the work on completion, and subsequent stress-tests made, gave every indication of excellent workmanship. It was by no means certain that the difference in the span-lengths between the girders was due to welding distortion. There was a web-splice at the knee, and the slightest difference in the clamping of that joint would have magnified the error many times at the end of the leg. On both sides of the girder, those differences might have been additive or compensating. In any case, such differences in span as did occur would cause little or no distortional stresses in the completed structure, since the holding-down bolts were adjusted to suit.

# CORRESPONDENCE
## ON PAPERS PUBLISHED IN
## FEBRUARY 1940 JOURNAL

Paper No. 5224.

## "Some Aspects of Aero-Hangar Design." †

By Archibald Milne Hamilton, B.E., M. Inst. C.E., and Edgar Basil Cocks, B.E., Assoc. M. Inst. C.E.

### Correspondence.

**Mr. C. O. Boyse** observed that the Authors had made a valuable contribution towards the study of aero-hangar design, the Paper dealing principally with the distribution of wind-pressures on the one hand and the stress-analysis of the portal frame on the other. Reference was also made at the end of the Paper to confirmatory tests which had been conducted at Hereford on the full-sized rib. No details were, however, given of those tests and, as they involved a number of unusual features of special interest, Mr. Boyse, who had been responsible for the carrying out of the work, would give a description of the various tests made.

It was perhaps important, before considering the tests themselves, to amplify the Authors' reference to the adoption of a factor of safety of $1\frac{1}{2}$ under hurricane conditions (namely, an 80-mile-per-hour wind). That basis of design required that the structure should withstand without failure or appreciable permanent distortion loads equivalent to the maximum working loads multiplied by the specified factor of safety. It thus followed that when the structure was subjected to its ultimate test loads no tension member would be stressed above its elastic limit and no strut would be loaded up to its crippling strength. In carrying out the tests on the typical hangar rib, which, as already stated, was designed to have a factor of safety of $1\frac{1}{2}$, the test loads were, therefore, calculated by adding a 50-per-cent. overload to the various wind-pressures and dead loads. It also followed from that interpretation of the expression "factor of safety" that at all stages of loading, up to at least the ultimate test values, the structure remained perfectly elastic and the distribution of stresses would be constant at all loads.

The tests carried out had been described in a Report, which contained

† Journal Inst. C.E., vol. 13 (1939–40), p. 305 (February 1940).

full information as to the deflexions of various parts of the structure under load*. The results indicated not only that the original design conditions were fulfilled, but also that the structure possessed a further margin of strength.

The plant employed for carrying out the tests was of special interest, as it was believed to be the only one of its kind in Great Britain. It was designed and installed in 1929 at the works of Messrs. Painter Bros., Limited, at Hereford, principally for the testing of lattice-steel transmission-line towers. It was capable of being used for destruction tests on towers up to 100 feet high, the loads being applied by means of electric winches and measured by hydrostatic dynamometers. The oil-pressures in those dynamometers were transmitted through flexible high-pressure pipes to a series of dials centrally situated in the test-house, so that a number of separately-applied loads might be controlled and measured accurately from one point. The testing-plant included three permanent towers, from which the loads were applied. Owing to the fact that the dynamometers were attached at various levels in relation to the pressure-gauges, corrections had to be made for the head of oil. That was, however, a simple process, and details of the corrections and of the manner in which they were made, were given in the test report referring to the hangar rib*.

**Mr. S. P. Wing**, of Denver, observed that the Authors, in presenting experimental data and a discussion of the effect of wind-loads on aero-hangar design, did much towards clarifying the nature of wind-loading, a subject which had an important influence on the cost of many light modern engineering structures.

In 1915 Mr. Wing had made measurements of the increase in wind-pressure above the ground on a wireless mast 492 feet high at Ballybunion, Ireland, using instruments lent by Sir Napier Shaw ‡. That study had been combined with one dealing with methods for deciding on "maximum winds for design purposes," by the application of frequency studies. The method, fully described elsewhere||, was based on the statistical viewpoint, and was the obvious one of studying the frequency of the maximum winds recorded near any locality; it yielded the equally obvious result that localities differed greatly in what might be called their "insurance risk" against a strong wind. When wind-loadings had no influence on a design, it was a matter of indifference if, for example, the

---

* "Transportable Aeroplane Shed. Test Report." *Prepared by* Callender's Cable & Construction Company, Ltd. London, 1936.

[This report and the three photographs accompanying Mr. Boyse's contribution have been filed with the Paper, and may be seen in the Institution Library.—Sec. Inst. C.E.]

‡ S. P. Wing, "Wind Pressures and the Design of Radio and High Transmission Towers." *The Electrician*, vol. lxxxvii (1921), p. 6 (1 July 1921).

|| S. P. Wing, "*Discussion on* Second Progress Report of Sub-Committee No. 31, Am. Soc. C.E. Committee on Steel, dealing with wind-bracing in steel buildings, Proc. Am. Soc. C.E. vol. 58 (1932), p. 1103.

building codes of London and Edinburgh specified the same wind-loading; when, however, the loading controlled the cost to an important degree, as in the case of aero-hangars and wireless towers, it was a matter of economic waste to build alike in the two localities if the "insurance risk" for heavy wind loading differed widely in the two locations.

The Authors, in *Fig. 17* (p. 322 §) presented what they called the wind-loading for "'normal' airtight conditions." When Mr. Wing first saw the results of aerodynamic testing of structural models, he, too, thought of the air-tight condition as "normal," and had been disturbed by the high suctions indicated as acting on roofs and lee walls. Further thought seemed to indicate that a compensating factor was usually present which made the "normal" loading of the individual walls much less. Ventilating engineers had found that even in rooms spoken of as "tightly closed" two or three air-changes an hour took place. For a hangar, the ventilating rate was probably as frequent as once in 10 minutes. The rate of ventilation depended to a considerable degree upon the wind-velocity and the corresponding pressure-differences. With positive pressure on one side of a building and negative pressure on the other, a pressure-gradient was set up which resulted in a constant flow of air into and out of the building through the walls, whether of wood, brick, or plaster, and through the cracks around doors and windows. Within the building the pressure was bound to be substantially constant. Since in most buildings from two-thirds to three-fourths of their superficial area was under an average negative pressure about equal to the average positive pressure, the pressure in the interior of the building would be negative. Using approximate figures from the data on the hangar presented by the Authors, 6,000 square feet had a mean pressure of +15 lb. per square foot and 14,000 square feet had a pressure of −13·5 lb. per square foot; for equal inflow and outflow the interior pressure was −5 lb. per square foot. From that the unit load acting against the windward side was 20 lb. per square foot, as compared with an average negative load of only 8·5 lb. per square foot acting on the lee side. The maximum intensity of loading would be about equal on both sides.

Mr. Wing agreed with the Authors that any possible loading caused by open doors, etc., should be provided for. However, he believed it to be consistent design to encroach on the safety factors for unusual possibilities, and that, in general, the side panelling should be designed to fail before the frame.

For structures in which wind-loading was an important part of the cost, investigation might show, as the Authors had suggested, that simple devices such as bursting panels or movable ventilators could be used to keep the interior at the most favourable pressure. Such an investigation might be worthy of research.

---

§ Page numbers so marked refer to the Paper (Journal Inst. C.E., vol. 13 (1939–40), p. 305 (February 1940)).—Sec. Inst. C.E.

The Authors, in reply to the correspondence observed that the tests referred to by Mr. Boyse were carried out on a full-sized rib. Such tests carefully executed with machinery capable of applying any desired system of loading with accuracy, were essential for checking the correctness of the calculations and of the various assumptions made. They formed an important part of the Authors' original recommendations for those hangars.

Further, as the wind-tunnel conditions were not always exactly simulated in practice, the Authors emphasized the desirability of making, as a check on the model-test, manometric measurements on a completed hangar during a strong wind. If the datum pressure was taken through a "static pressure-tube" clear of the influence of the building, a comparison might be made with the figures obtained in the model-experiments and the effect of leakage on the internal pressure might be determined. That information would be very useful in future designs of sheds of the same class. A slight economy of material could represent the saving of a large sum of money if such sheds had to be provided for service-use on a national scale of supply. Even for a single shed the saving would, generally, easily repay the cost of the tests, and, at the same time, would give the engineer complete confidence in his work.

Mr. Wing had pointed out that owing to the non-airtightness of hangars the effect of leakages into the building on the pressure side and out of the building over the area of covering under suction, would result in an internal pressure that would be different from the datum adopted for the "airtight" condition (*Fig. 17*, page 322 §) which was virtually the basis of the model-tests carried out by the National Physical Laboratory.

Mr. Wing had calculated that under the average pressure and suction on the areas over which those respectively operated, as determined from *Fig. 17*, the pressure inside the hangar would be reduced on account of leakages by 5 lb. per square foot. The calculation assumed that the leakage areas were proportional to the area of the covering. That might or might not be so, but in the absence of abnormal leakage-vents, such as doors with large gaps between sliding components, the assumption would seem to be tenable. As, however, such a reduction in the internal pressure would reduce the maximum bending moments considerably (as might be seen from a comparison of *Figs. 18* and *26*, and *Figs. 19* and *27*), it would have to be made with caution from the design point of view.

It was fully realized by the Authors that the datum adopted by the National Physical Laboratory (page 308 §) did not necessarily represent the only possible one in practice; hence the use of the word "airtight" in *Fig. 17*. In fact, under certain conditions the internal pressure could lie anywhere between the extremes shown in *Figs. 20* and *24*.

A reduction of the internal datum pressure of *Fig. 17* of the order of 5 lb., as suggested by Mr. Wing, would reduce the bending moments, and

---

§ *Ibid.*

would allow the employment of lighter structural members. It might in fact be quite possible, as suggested by Mr. Wing, to install ventilators that would ensure a definite reduction of internal pressure, and that would be a distinct novelty in design. Such ventilators would require to have ample ventage-area and to be of such a shape and location that the internal pressure could be reduced to some predictable value. It was suggested that ventilators almost flush with the roof-covering would be the most suitable type for transmitting the external manometric pressure to the interior. The values given in *Fig. 17* would seem to indicate that the centre of the roof would be a suitable location for the ventilators. The suction on that line was more or less independent of the direction of the wind. The effect of the ventilators should, of course, be checked by experiment.

---

Paper No. 5202.

## "The Deterioration of Concrete in Contact with Sewage." †

By Solomon Simon Morris, B.Sc., Assoc. M. Inst. C.E.

### Correspondence.

**Mr. E. J. Guild** observed that there might be a possibility of condensation from the sewage forming on the upper parts of pipes or tanks. Such a condensate would have all the corrosive effects of distilled water, as well as those due to any dissolved gases evolved by decomposition of the sewage. What would be the $p$H-value of such a condensate, what $p$H-value would be most favourable to the growth of *Spirrilum desulphuricans*, and what would be the most inhibiting value ?

**Mr. R. H. H. Stanger** was particularly impressed by the implication that the corrosion of the concrete was due to the formation of sulphuric acid formed by the bacterial reduction of sulphates. In his experience, soluble mineral sulphates had a much greater corrosive effect on concrete than had sulphuric acid. If that were agreed, any method of prevention of corrosion which aimed at the prevention of the bacterial reduction was of little use. It was the action of the sulphates themselves which had to be prevented.

† Journal Inst. C.E., vol. 13 (1939–40), p. 337 (February 1940).

*** **Mr. L. C. Woolley** observed that the extract from the Author's communication to The Institution represented to a large extent a Report on a preliminary investigation into the deterioration of concrete in the sewerage system of Cape Town carried out by him in March 1931, and should not be taken as representing the true position to-day. From the observations then made certain conclusions were drawn and remedial measures were proposed and carried out. Mr. Woolley wished to show which conclusions had been proved to be correct by subsequent investigation, by commenting on the results of the remedial measures carried out. He would also mention that reference to the subject had been made elsewhere.*

The very brief description of the physical characteristics given in the Paper might be misleading to those who had not become acquainted with the phenomenon. Corrosion, so far as investigations in Cape Town indicated, was confined to materials or structures in which cement or lime was an ingredient. Glazed sewer-pipes and bricks were unaffected. Concrete and the jointing of bricks or tile-work were the materials affected, and corrosion took place only when the materials were not continuously submerged.

Above high-water mark the initial visible stage of corrosion of both the concrete and the jointing was the formation of a white powdery deposit similar to efflorescence. In the case of concrete that deposit thickened, became continuous, and formed bulky flakes which had no strength or coherence and consequently broke away under their own weight, leaving a soft spongy surface exposed. Corrosion continued at an increased rate owing to the porosity of the exposed surface, and further flakes and pieces of coarse aggregate became detached from the structure. In the case of

---

*** Mr. Woolley's contribution has been abstracted from a lengthy communication describing additional tests and giving details of the chemical analysis of the mechanism of corrosion; the MS. and illustrations may be seen in the Institution Library.—SEC. INST. C.E.

* R. F. Goudey, "Odor Control by Chlorination." California Sewage Works Journal, vol. 1 (1928), (December 1928). [Reprinted in *Western Construction News*, San Francisco, vol. 4 (1929), p. 16 (10 Jan. 1929).]

It was stated that the 26-mile trunk outfall sewer was badly affected in Orange County, Cal., and that chlorination eliminated corrosion.

A. F. Pistor, "Effects of Sewage Gases on Concrete." *Sewage Works Journal*, vol. 7 (1935), p. 697 (July 1935).

At Canton, Ohio, in 1934, an interceptor constructed from vitrified tile failed owing to corrosion of the joints.

E. G. Studley, "Experimental Ventilation of the North Outfall Sewer of the City of Los Angeles," *Sewage Works Journal*, vol. 11 (1939), p. 264 (March 1939).

It was described how the disintegration of the 55-mile north outfall sewer of the City of Los Angeles was held in check by forced ventilation.

F. M. Lea, "Deterioration of Concrete owing to Chemical Attack." Journal Inst. San. E., vol. 39–40 (1935–36), p. 185.

L. H. Enslow, "Sewage Chlorination for the Protection of Masonry Sewers against Deterioration." *Water Works and Sewerage*, vol. 67 (1920), p. 306 (September 1920).

the jointing, the second stage was a swelling of the joints owing to the increased volume of the corrosion-product. That expansion might take place outwards, in which case the swelling reached a certain point and then broke away under its own weight, leaving a soft putty-like substance in the joint. If the expansion took place sideways as well as outwards, sufficient pressure was set up to shear off the faces of the adjoining bricks.

Corrosion also took place between the levels of high water and low water, and under certain conditions corrosion at, and immediately below, high-water mark was extremely severe. During low water, corrosion occurred between high-water level and low-water level, and when the sewage flow increased the corrosion product was washed away. That corrosion and subsequent erosion resulted in removal of the matrix, causing exposure and final breaking away of the aggregate. The characteristic appearance showed the coarse aggregate protruding from a soft dirty-white putty-like substance. The actual corrosion was, on that area, often masked by a deposit of slime and organic matter from the sewage. When a short peak flow occurred at long intervals the portion of the structure immediately below high-water level showed the greatest corrosion, but when the peak flows were of longer duration, and occurred more frequently, maximum corrosion occurred above high-water level.

After the submission of the Report covered by the Paper, further examinations confirmed in general the previous observations, but in addition revealed that :—

(*a*) pneumatic ejectors were a distinct improvement on pumping-stations, as no signs of attack were found at any of the discharge-pipes where they entered the interceptor, although some had been in operation for more than 30 years;

(*b*) in every case where reduction in velocity of flow occurred in the reticulation, causing deposition of silt, corrosion occurred;

(*c*) corrosion did not occur where ventilation was sufficient to maintain unsubmerged portions of the structure free from condensed water.

In order to study the corrosive action further, a series of tests was inaugurated, the first of which consisted of the test-patches in the pump-well of the main pumping-station mentioned in the Paper. The other tests were :—

(i) Approximately 16 square yards of surface on the east wall of the pump-well were chiselled out and 3 : 1 Ciment Fondu plaster ½ inch thick was added. That was coated with "Inertol." A strip approximately 12 inches wide and 8 feet long was chiselled out and plastered on the west wall opposite the above-mentioned area, and left unpainted.

(ii) A series of test-slabs were hung in the interceptor, for observation both before and after chlorination.

(iii) Three specially-made pipes were placed in the 24-inch gravity main taking tank-effluent at the Athlone disposal-works. They were fitted with multiflex joints so that each pipe could be removed and inspected periodically. Pipe No. 1 was made of type-4 mix * Ciment Fondu. Pipe No. 2

* 2 parts of stone, 2 parts of sand, and 1 part of cement.

was a Portland-cement pipe with a Ciment Fondu lining. Pipe No. was a Portland-cement pipe which, after curing, was treated with paraffi wax.

Physical examination of the 12-inch square test-patches at the ma pumping-station, after that described by the Author, confirmed th superiority of Ciment Fondu over some other cements, but indicated th wire-brushing the surface before application did not give a satisfacto bond. Sand-blasting had been used extensively in America for ensuring good bond between old and new concrete, and it was suggested that th method might be used with advantage in removing corroded concre before re-plastering.

Physical examination of the two "Inertol"-coated Ciment Fonc strips indicated that less corrosion had taken place on the painted surfa than on the unpainted surface. Microscopic examinations of samples the deposits on the uncoated Ciment Fondu showed them to be main

TABLE II.

| | Pipe No. 1. | Pipe No. 2. | Pipe No. 3. |
|---|---|---|---|
| Material. | Ciment Fondu 2:2:1 mix. | Portland cement, lined with 2:2:1 Ciment Fondu. | Portland cement impregnated with paraffin wax. |
| Maximum corrosion after $14\frac{1}{2}$ months' use: inch | $\frac{9}{64}$ | $\frac{13}{64}$ | $\frac{7}{64}$ |
| Life of pipe, based on first $14\frac{1}{2}$ months' service: years | 10·5 | 7·9 | 15·2 |
| Maximum corrosion after 2 years' use: inch | $\frac{18}{64}$ | $\frac{17}{64}$ | $\frac{31}{64}$ |
| Life of pipe, based on corrosion during last $9\frac{1}{2}$ months of test: years | 7·5 | 16·4 | 3·1 |
| Life of pipe, based on corrosion during entire 2 years of service: years | 9·1 | 10·3 | 5·8 |

gypsum, whilst the deposits on the coated Ciment Fondu consisted c sulphur and gypsum. Bacterial threads resembling *Beggiatoa* wer observed, as well as numbers of bacilli and vibrios. Chemical analysi in conjunction with microscopic examinations, showed that a high per centage of free sulphur was found on "Inertol"-coated Ciment Fondu whereas the deposit on the uncoated Ciment Fondu contained mor than 90 per cent. of gypsum. That confirmed the physical observatio that less corrosion had taken place on the "Inertol"-coated strip.

Table II gave the extent of maximum corrosion occurring in the thre test-pipes, and also the computed life of each pipe, based on the corrosio occurring during the period that the pipes were under observation. Fron examination of pipes which had failed, and from a study of the aggregat

used in the concrete, it was assumed, in computing the life of a pipe, that failure would occur when the thickness of the pipe was reduced to $\frac{1}{4}$ inch.

It was of interest to compare those computations of the life of the pipes with computations based on the corrosion of concrete in the sewers of the Cairo main drainage scheme. In Mr. A. O. W. D. Pinson's Paper*, the average depth of corrosion in $7\frac{1}{2}$ years, scaled from the diagrams, was $3\frac{3}{4}$ inches, giving an average corrosion-depth of $\frac{1}{2}$ inch per year. The "Hume" pipes at the Athlone disposal-works were $1\frac{1}{2}$ inch thick, and thus would have a life of only 3 years under those conditions.

The results indicated a slight advantage in favour of Ciment Fondu as a material, but taking the measurements as a whole, the Ciment Fondu pipe No. 2 was definitely superior. The corrosion of $\frac{13}{64}$ inch occurred at only one point in the section of No. 2 pipe, and was probably due to a porous patch of cement; the subsequent corrosion occurring in the second year might represent more accurately the resistance of Ciment Fondu. No. 3 pipe stood up very well during the first period, but during the second period the resistance of the paraffin wax broke down and excessive corrosion occurred. That ruled out paraffin wax as a protective coating for concrete against that form of corrosion.

At the time of the second examination the conditions under which the pipe-line operated were altered. The outlet was changed to allow free discharge, and at intervals of 50 feet the pipes were replaced by open channels 7 feet and 2 feet in length. In addition, an 18-inch cast-iron pipe-line was laid to relieve the load on the concrete pipe-line.

Examinations carried out recently had shown that slight corrosion occurred at each open channel following a 2-foot opening. In other words, an opening of 2 feet in a length of 100 feet did not give sufficient ventilation to prevent corrosion. Where there were 7-foot openings in a length of 100 feet the pipes appeared sound.

From consideration of the observations in the light of the mechanism of corrosion, the following represented a summary of the important factors relating to the corrosion of concrete :—

(A).—*Factors causing and aiding corrosion.*

(1) Retention in pump-wells.
(2) The deposition of silt and organic matter in sewers.
(3) Seeding of fresh sewage with stale organic matter.
(4) Agitation.
(5) Condensation on unsubmerged portions of the structure.
(6) Summer conditions, including higher temperatures, lower flows, and longer retention-periods.

* "Cairo Main Drainage Extensions." Minutes of Proceedings Inst. C.E., vol. 231 (1930–31, Part 1), p. 130.

(B).—*Factors preventing or retarding corrosion.*

(1) The use of pneumatic ejectors.
(2) The total exclusion of air.
(3) Complete access to the air.
(4) Increased ventilation.
(5) Chlorination.
(6) The use of Ciment Fondu.
(7) The use of "Inertol," or some other bituminous coating.

A careful survey of the sewerage-system was made to ascertain wheth any changes in operation could be effected to prevent or to retard corrosio As a result of trials, the following alterations had been made in regard t the operation of the system. Sumps at sub-pumping stations were clean out twice weekly instead of weekly; the permissible flow to the disposa works was increased, and one of the storm-water tanks was utilized as balancing-tank to prevent back-flooding, enabling the sewer to be clean out; and at the disposal-works the sedimentation-tank effluent carri was altered to give free discharge, and large openings were provided ever 50 feet. Attention had also been directed to means of preventing corrosion A chloronome was installed on the Maitland sewer to feed chlorine at point immediately after the discharge of the rising mains from the sub pumping stations, and badly-damaged structures connected with tha sewer were reconstructed, Ciment Fondu being used throughou Recent examinations indicated that some of the reconstructed wor showed no sign of attack after approximately 2 years' service. Th sumps of the main pumping station were reconditioned throughout wit Ciment Fondu, coated with "Inertol." Later, a chloronome wa installed, primarily to counteract odours at the station and at the disposa works; it also had the effect of retarding corrosion. An inspection of th sump of the main pumping-station carried out in 1940 indicated that th improved operation of the sub-pumping stations, the prevention of back flooding in the main sewer, and chlorination, had retarded corrosion to considerable extent. The upper portions of the sump were sound, an corrosion to the extent of approximately $\frac{1}{4}$ inch was observed only betwee high- and low-water level. That indicated that hydrogen-sulphide pro duction had been reduced to such an extent that the amount of gas presen was below that necessary to saturate the sewage, and that consequentl no appreciable liberation of hydrogen sulphide had taken place. Th reconstructed work appeared sound, except for certain points, strangel enough between high- and low-water level, where the bond was not good

Similar measures had, however, failed at the Muizenberg pumping station, put into operation in 1934. On account of odour-complaints attempts to provide adequate natural ventilation had had to be abandoned and in view of the apparent resistance of the "Inertol"-coated Cimen Fondu test-strip at the main pumping-station, the Muizenberg sump wa

given two coats of "Inertol." Owing to lack of ventilation in the Muizenberg sump, however, the painting having been done after the station had been put into operation, it was impossible to get the walls thoroughly dry before applying the "Inertol." Examination in 1935 showed that there was very little bond between the "Inertol" and the "Ferrocrete," and that the concrete of the sump had corroded slightly. In 1938 a further examination was carried out, and two 12-inch by 6-inch cement test-slabs of 3 : 1 Portland cement were placed in the sump : one was suspended so as to be nearly always submerged, whilst the other was suspended at such a level as to be submerged only at high water. After 3 months and after 11 months slight corrosion was observed, but during the subsequent 12 months the action appeared to have become intensified. The slab which was submerged most of the time was sound, whilst the other slab was considerably corroded. At the last inspection, in January 1940, the walls of the sump showed the typical flaking putty-like deposit, and had softened to a depth of 1 inch in places.

The observations on that station indicated that the use of resistant materials was no safeguard against that form of corrosion, unless in addition the cycle of happenings leading to the formation of sulphuric acid was broken or retarded by one or more of the means previously mentioned.

Mr. Woolley would like to acknowledge the collaboration of Mr. A. Abbott, B.Sc., Assistant Chief Chemist at the Athlone disposal-works, with whom he was closely associated, by courtesy of the City Engineer, Cape Town, for the greater part of the investigation described.

**The Author,** in reply, wished first to thank Mr. Woolley for his observations. The purpose of the Paper had been to demonstrate to engineers the fact that sulphur bacterial action could be counteracted, but it had given only a very brief outline of the problem. The Author considered that a most authoritative and valuable contribution had been made by Mr. Woolley to the scanty literature on the subject.

He was particularly indebted to Mr. Abbott for assistance in replying to some of the queries raised in the Correspondence.

Mr. Guild's observation regarding condensation forming on and attacking the concrete had been actually borne out in practice. Such condensation was mentioned by Mr. Woolley on p. 533, *ante*, and on p. 535, *ante*.

Corrosion occurred on the upper parts of sewers or appurtenant works whenever the ventilation, whether forced or natural, was inadequate to maintain those parts completely dry. Condensation alone, however, was not enough to precipitate corrosive action. It was essential that hydrogen sulphide be present, and it was through the secondary solution of hydrogen sulphide that the final changes which led to corrosion actually took place. Attempts to eradicate corrosion by ventilation had failed when due cognizance had not been taken of that important factor. Such an instance

had been recorded in the abortive efforts to reduce corrosion in the mai drainage of Cairo in 1918[1], where one large fan had been installed to exhau sewer-air from a trunk sewer about 8 miles long, only one air inlet bein provided at the head of the sewer. That was inadequate to prevent con densation in such a length of drain, whilst the reduction in pressure in th sewer caused the liberation of a constant stream of hydrogen sulphid which, coming in contact with the condensate on the upper portion of th sewer, set up an ideal combination for corrosion.

The final process of corrosion was the formation of sulphuric acid i the thin film of condensed moisture which contained the dissolved hydroge sulphide. The main agents responsible for that essential part of the proce of decay were considered to be sulphur bacteria.

Generally, however, owing to its relatively small quantity, the actio of such a condensate would be slight and it could hardly be compare with the action on concrete actually submerged in distilled or chemical pure water.

The *p*H-value of that condensate would depend upon several variabl factors difficult to determine, of which the nature of the gases in the sewe the extent of sulphuric acid production, and the consequent action on th concrete were some of the most important. Actually no definite *p*H-valu could be laid down.

The organism *Spirrilum* or *Vibrio desulphuricans* developed strongl at *p*H-values between 6·9 and 7·5, and presumably the optimum *p*H-valu for that particular bacterial development lay between those two values.

There was, however, no question of *Spirrilum* or *Vibrio desulphurican* acting in the condensate on the upper parts of sewers or tanks, since tha organism, which reduced the sulphates to hydrogen sulphide, functione only under anaerobic conditions below the surface of the sewage itself.

Mr. Stanger's contention that soluble mineral sulphates had a corrosiv effect upon concrete greater than that of sulphuric acid would be tru only when the sulphuric acid solution was of a considerably weaker con centration than the sulphate solution. In equal concentrations the cor rosion due to sulphuric acid would exceed by far that induced by a sulphat solution. In any case, the deterioration of concrete in contact with sewag was definitely not brought about by the direct action of soluble sulphate Chemical analysis indicated that it was due ultimately to sulphuric aci produced by sulphur bacterial action. The quantity of soluble sulphate present in sewage was usually too small to cause direct corrosive actio The highest sulphate-content encountered in Cape Town sewage was le than 20 parts per 100,000, or 0·02 per cent. Most authorities agreed tha before concrete pipes would be attacked the sulphate-content in solutio would have to exceed between 0·05 and 0·1 per cent.

---

[1] A. O. W. D. Pinson, "Cairo Main Drainage Extensions." Minutes of Procee ings, Inst. C.E., vol. 231 (1930–31, Part 1), p. 114.

Actually, in the case of cements immersed in sulphate waters, the expansion and disintegration were caused by the formation of gypsum ($CaSO_4.2H_2O$) and of calcium sulpho-aluminate, which crystallized with the formula $3CaO.Al_2O_3.3CaSO_4.31H_2O$. The latter salt was not found to form under the acid conditions existing in the corrosion under discussion. Moreover, conclusive evidence that the corrosion was not produced by sulphates in solution was provided by the fact that distintegration took place on those portions of the sewerage systems which were either entirely or intermittently unsubmerged, and never on those portions which were continually under water.

The Author wished to thank Mr. W. S. Lunn, Assoc. M. Inst. C.E., City Engineer, Cape Town, for permission to publish the Paper, and he hoped that by bringing the problem of sulphur bacteria, its action, treatment and prevention to the notice of engineers throughout the world a step had been taken towards the complete eradication of that costly and highly-virulent agent of destruction.

---

# CORRESPONDENCE
## ON PAPER PUBLISHED IN MARCH 1940 JOURNAL.

---

Paper No. 5217.

## "The Dragline Excavator." †

By William Barnes, M. I. Mech. E.

### Correspondence.

**Mr. C. B. Bailey** observed that he was concerned with the larger types of excavator, but unfortunately the majority of digging that his firm had to meet was of such a nature that dragline excavators could not face it; much had been made of the recent developments in draglines tending towards the handling of heavier rocks, but he thought that it would be generally accepted that such rocks had to be expensively blasted before the dragline type of machine could deal with them.

† Journal Inst. C.E., vol. 14 (1939–40), p. 8 (March 1940).

It appeared to him that those taking part in the Discussion had confine their remarks to the smaller type of contractors' unit, and whilst the drag line had brought about great improvements in the methods of handlin ordinary material, the iron-ore industry had demanded even furthe developments in large excavators and their application. During the las 6 or 7 years facilities for electrical operation had reached country district and the intensive development of large excavators had probably com about on that account. Until that time the principal prime mover for th larger type of machine had been steam, with consequent excessive labou charges in the handling of coal; for example, a machine of nothing lik the magnitude of present-day units needed two firemen per shift to hand a consumption of 10 tons for that period. The installation of strippe of up to 1,000 horsepower had resulted in the development of ore reserv under overburden, the working of which could previously only have bee considered by underground methods. Draglines had not been installed fo those arduous duties, however, but had been mainly employed for th stripping of lighter types of overburden, and, in some cases, prelimina stripping of the lighter measures. The general view of the industry wa that the walking dragline was a progressive step in the method of trave giving greater ranges and larger-capacity buckets on account of the redu tion in the bearing pressure of the machine; he would, however, stress tha in his view the quality of the digging was the prime consideration as t whether a dragline machine could operate efficiently and/or economically.

Draglines had operated for Mr. Bailey's firm in overburden up to abou 15 feet deep, and the quality of the digging had not been severe excep for casual laminated beds of limestone. He thought that it would b agreed, however, that the results obtained were remarkable, and th operating costs shown in Table VI fully supported the figures given b the Author. In considering the Table, it should be borne in mind tha the machine concerned had been in constant operation for some 6 years and had, therefore, got over the preliminary efficiency expected from new machine during, say, its first 2 years of life.

The item of cost specially noted for drag-ropes was worthy of attentio and he thought that most dragline users were dissatisfied with the life o those ropes. His firm had carried out numerous experiments with (a) heavy and light specially-designed swivels; (b) the most suitabl tensile strength for the wire used in the ropes; and (c) the constructio of the ropes. The tendency of the swivels was to unreave the rope owing t the excessive spin which appeared to take place in drag-ropes. With regar to (b), they had come to the conclusion that the lower-tensile-strengt ropes with a strength of, say, 20/100 tons per square inch), gave better lif than those with a higher tensile strength, but rope-makers were reluctan to go to a figure lower than that given. Such experiments as his firm ha carried out in regard to (c) had given no improvement upon the standar supplied by rope-makers.

Generally, the costs shown in Table VI covered all operational work, including the cost of two men around the machine, whereas the item for repairs, overhauls, etc., included all de-carbonizing (carried out every 6 months). No amortization and on-cost was included; however, the experience of his firm was that the useful life of a machine of the type concerned was about 15 years.

Mr. Bailey agreed that the optimum efficiency in the design of the bucket had not yet been reached, but the various conditions in which draglines operated caused considerable divergence of opinion; in the case of his firm it was found that the angle of the teeth to the bottom of the bucket (strictly, relative to the mouthpiece) was an important point, and on that account they had found that, provided that that angle was correct

TABLE VI.

Year: 1939.

Type 43-B dragline, excavating surface soil and sub-soil of rubbly ironstone in separate operations, and depositing for restoration after extraction of iron ore.

Hours worked: 1,940¾. Cubic yards moved: 112,454.

Capacity of bucket: 1½ cubic yard. Engine: 6-"Varn" (134-horsepower) diesel.

| Item. | Cost per hour. | Cost per cubic yard. |
|---|---|---|
| 1. Labour operating | 2*s*. 3*d*. | 0·47*d*. |
| 2. Repairs, overhauls, ropes, spares, etc. | 3*s*. 3*d*. | 0·67*d*. |
| 3. Fuel | 10*d*. | 0·17*d*. |
| 4. Other costs | 1*s*. 0*d*. | 0·20*d*. |
| Total: | 7*s*. 4*d*. | 1·51*d*. |
| *Note:* Drag-ropes included in item 2 | 1*s*. 3½*d*. | 0·27*d*. |

for the type of digging, it was possible to lengthen the teeth with efficient results: in the case of one bucket they had increased the teeth-length by 3½ inches, resulting in a considerable increase in production. The same remarks applied to the type of steel used in the construction of the bucket; most manufacturers were using manganese steel for the mouthpiece, but in Mr. Bailey's view that was not by any means necessary in the majority of jobs. The virtue of manganese steel was that face-hammering tended to harden the face, and so resisted wear, but in many cases the material dug was of a soft nature, and, therefore, no face-hammering occurred, with the result that the manganese steel lost its beneficial characteristic; in fact, his firm had found that mild steel was more suitable for the mouthpiece, particularly as it could easily be repaired by arc-welding. It had also been found that mild-steel teeth of the longer type could be re-sharpened by forging, thereby adding life to the teeth.

Mr. Bailey was very interested in the remarks made in the Discussion

in regard to the elasticity of steam-driven machines, but that extend[ed] the issue very much further than the Author had done. The use of War[d] Leonard control on the larger type of machines had, however, produce[d] characteristics very closely approaching those of steam-driven unit[s]. Nevertheless, the diesel engine had a virtue which could not be disregarde[d], namely, that of giving heavy torques at low speeds.

**Mr. Harry Fairclough** was particularly interested in Mr. Ireland['s] comments, and in the Author's statement on p. 12 § in which he sai[d] "Users are recognizing that the lighter the bucket, consistent with i[ts] ability to dig, the greater the efficiency or pay load, and, if a larger outp[ut] is required or can be dealt with, the increase in output quickly pays for [a] new bucket." As the Author pointed out on p. 50 §, Mr. Fairclough ha[d] purchased Ruston's first dragline in 1918, the first to be built in Grea[t] Britain. He had not, however, used the bucket supplied with the machin[e], but had employed a specially light bucket of his own design. That buck[et] had part of the back cut away so that there was no corner for the stick[y] "bungum" to adhere to. It also let the "cut" run through the buck[et] until the bottom of the bucket was polished, and prevented adhesion of th[e] material. That bucket had doubled the capacity, and was from one-ha[lf] to one-third of the weight of the bucket offered with the machine. Th[at] specially light bucket dug 250,000 cubic yards before breaking up, so tha[t] it more than "earned its keep."

Theoretically, he maintained that it was necessary to have as man[y] types of dragline buckets as of hand-shovels, grafting tools, coal-spade[s], square-mouthed spades, pointed gravel- or macadam-spades, coal-trimm[er] spades, snow-shovels, and garden-spades. The ordinary general-purpo[se] builders' spade, which had been evolved by trial and error, was capable [of] digging and holding 3 times its own weight in earth; therefore a draglin[e] bucket weighing 574 lb. should carry the 1,722 lb. of earth that it would di[g]. His bucket would do that, and had been designed to that theory.

The usual heavy-type 8-cubic-foot bucket, weighing 604 lb. wit[h] teeth, was a pick as well as a spade, but owing to the strong way it wa[s] constructed it had a resisting area on the bucket side, bottom, teeth, an[d] gudgeon brackets, that in his opinion wasted a good deal of energy. O[n] the "H.F." light 16½-cubic-foot bucket the resisting cutting-area wa[s] 78 square inches, and on the Ruston-Bucyrus bucket of 8 cubic fee[t] capacity it was 122½ square inches. The ultra-light buckets that he ha[d] made and used on Ruston Nos. 4 and 10 R.B. machines since 1926, wit[h] capacities of 16½ cubic feet, weighed 574 lb., and were fitted with ½-inc[h] drag-chains. He had used them on a 35-foot jib set at an angle which wa[s] stable for the machine (that being left to the driver). During the las[t] 14 years Mr. Fairclough had worn to destruction only two buckets: the[...]

§ Page numbers so marked refer to the Paper (Journal Inst. C.E., vol. 14 (1939–40), p. 8 (March 1940)).—Sec. Inst. C.E.

had been repaired with new cutting-lips as required. He had also found that the $\frac{1}{2}$-inch drag-chains on the $16\frac{1}{2}$-cubic-foot bucket lasted longer than the $\frac{5}{8}$-inch chains on the Ruston-Bucyrus 8-cubic-foot bucket, especially at the coupling link.

With the Ruston-Bucyrus No. 10 R.B. machine, using Mr. Fairclough's light bucket, one driver, for a wager, loaded 156 tons of burnt red ore in 75 minutes into thirteen wagons without a shunter, and loaded forty-five wagons, equal to 540 tons, in 7 hours. The same driver loaded 570 cubic yards of loam into road-wagons in 11 hours with the same light bucket.

**Mr. A. F. Holden** observed that, so far as The Stanton Ironworks Company, Limited was concerned, it was at the suggestion of the Author that a steam-driven dragline had been first introduced for stripping ironstone in 1925, in an ironstone quarry with shallow overburden. That machine first removed the 9–12 inches of soil before removing 3–6 feet of subsoil, and then replaced them in their correct position on the ground from which the ironstone had previously been extracted. That work had formerly been done by hand-labour at a considerable cost. It was found that the dragline did the work better than hand-labour, as all the soil was preserved, and it proved that the restored land was better from an agricultural point of view than it had been before the ironstone was worked, because a farmer ploughed his land only from 4 to 6 inches deep, leaving as much soil again which was never brought into cultivation. When the soil was removed with a dragline the whole of it was lifted, aerated, and mixed before it was replaced, and soil was again brought into cultivation that contained unconsumed manurial matter which materially benefited the crop, which could be sown immediately.

Mr. Holden's firm at present had six draglines employed on that work, and they had another ten machines stripping, soiling, and also loading ironstone. That was found to be the most economical method of work where it was possible; the sizes of buckets ranged from $\frac{1}{2}$ cubic yard to 4 cubic yards capacity, with booms of up to 75 feet in length, on steam, diesel, and electric machines.

Mr. Holden had accompanied the Author on a visit to Belgium to see the machine illustrated in *Fig. 11* (facing p. 26 §) working on the Albert canal, as a result of which four walking draglines had been ordered by Mr. Holden's firm. The first of them was already working. It was the first of its type to be introduced into Great Britain, its main particulars being: bucket-capacity, 3 cubic yards; length of boom, 135 feet; and working weight, 143 tons. Its task was to soil and strip up to a depth of 40 feet, and to load out the bed of ironstone into wagons running on a tramway on the surface alongside the machine. If that proved to be successful, Mr. Holden thought that the machine would be the forerunner of a large number of walking draglines, which would be used for stripping to depths

§ *Ibid.*

which, up to the present, had been considered impossible; at the sam time it was possible to soil and level the surface of the ground on which th overburden was deposited, so as to discontinue the practice of leavin unsightly heaps, which existed in some parts of the country.

**Mr. B. W. Huntsman** noted that reference had been made to th Salonika Plain reclamation-works. On those works seventeen draglines, sizes from ¾ cubic yard up to 6 cubic yards, had been used for the excavatio of canals and the formation of embankments. Some particulars of thos draglines, for comparison with the figures given in the Paper, were show in Table VII.

TABLE VII.

| Draglines. | Horsepower per cubic yard. | Horsepower per ton weight. | Weight: tons per cubic yard capacity. | Ground-pressure: lb. per square inch. | Average output: cubic yards per hour per cubic yard of bucket capacity. |
|---|---|---|---|---|---|
| Small . . | 84–88 | 2·4–3·0 | 28–36 | 11–13 | 46–72 |
| Large . . | 63 | 0·83 | 76 | 22½ | 31 |

Mr. Davidson had suggested that it would have been more convenien if the working costs of those machines had been given in pence instead c in cents. Unfortunately, that was not possible, as the exchange-rate of the Pound sterling, the United States dollar, and the Greek drachm had fluctuated considerably throughout the period of the work. In an case, comparison of the costs of work in different countries and at differen times was apt to be misleading without full knowledge of the costs c labour, fuel, and lubricants, and of all the local conditions of the work The details of the comparative operating costs of diesel and steam machine on the Salonika works could be seen from Table VIII.

TABLE VIII.

| Draglines. | Bucket-capacity: cubic yards. | Fuel. | Unit costs: cents per cubic yard. | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Fuel. | Lubricants, waste, etc. | Spares and repairs. | Labour on machine. | Labour on repairs. | Total. |
| ON LAND | | | | | | | | |
| Diesel . . . | 2 | Diesel oil | 0·20 | 0·25 | 0·80 | 0·83 | 0·21 | 2·29 |
| Steam . . . | 2½ | Coal | 1·73 | 0·26 | 1·04 | 1·42 | 0·43 | 4·88 |
| FLOATING | | | | | | | | |
| Diesel-electric | 3½ | Diesel oil | 0·40 | 0·38 | 0·77 | 1·51 | 0·19 | 3·25 |
| Steam . . . | 1¾ | Coal | 2·00 | 0·28 | 0·96 | 1·63 | 0·13 | 5·00 |

The lower operating costs of the diesel machines resulted primarily from the lower costs of diesel oil, and secondarily from the lower cost of labour on the machines. As could be seen, the cost of excavation under water was relatively more expensive than on land. The diesel draglines on land had an hourly output per cubic yard of bucket-capacity 12½ per cent. greater than that of the steam machines. The time lost in washing-out and greasing the steam machines was 13 per cent., as compared with 7 per cent. lost time in greasing the diesel machines. The average life of drag-ropes was 3–5 weeks; hoist ropes lasted 4–7 weeks. The use of timber mats for the 6-cubic-yard draglines increased the cost of excavation by about 10 per cent. in wet weather and on bad ground.

Mr. Brocklebank had commented on the fact that the ¾-cubic-yard diesel dragline had actually shown lower excavation-costs than had the 2-cubic-yard diesel machines. To compare the two types of machine

TABLE IX.

| Diesel machine. | Output: cubic yards per month. | Percentage of maximum. | Cost: $ per month. | Unit costs; cents per cubic yard. | | | |
|---|---|---|---|---|---|---|---|
| | | | | Fuel and lubricants. | Repairs, etc. | Labour on machine. | Total. |
| 2 cubic yards (actual) . | 33,500 | 47 | 770 | 0·45 | 1·01 | 0·83 | 2·29 |
| ¾ cubic yard (actual) . | 22,300 | 76 | 380 | 0·36 | 0·62 | 0·73 | 1·71 |
| ¾ cubic yard (adjusted) . | 13,800 | 47 | 356 | 0·40 | 1·00 | 1·18 | 2·58 |

fairly required an examination of the conditions under which each was working. The excavation from the 2-cubic-yard draglines had generally been used in the formation of embankments; the ¾-cubic-yard machine had been employed for drainage channels where the spoil could be dumped without careful placing. The figures for the 2-cubic-yard draglines were the average of 53 machine-months; those for the ¾-cubic-yard referred to only 6 months' work*. The time lost per month in moving the 2-cubic-yard draglines from place to place was greater than in the case of the ¾-cubic-yard machine. An indication of the conditions under which the machines worked could be obtained from the fact that the average output of the 2-cubic-yard draglines was only 47 per cent. of the best month's output, whereas the ¾-cubic-yard machine had an average output of 76 per cent. of its best month's output. A rough comparison of the probable unit costs of the two types of machine when working under similar conditions could be made by adjusting the figures for the small machine, as shown in Table IX.

* B. W. Huntsman, "The Salonika Plain Reclamation-Works." Journal Inst. C.E., vol. 5 (1936–37), p. 280 (March 1937).

The main reason for the lower unit cost of the larger machine, compare[d] with the adjusted cost of the smaller machine, was the lower labour cost p[er] cubic yard. No fair comparison of capital costs was possible on accoun[t] of the different conditions which existed at the different times when th[e] machines were purchased.

In Table X the operation-costs of a diesel-electric multi-buck[et] excavator were compared with those of the 6-cubic-yard diesel-electr[ic]

TABLE X.

| Diesel-electric machines. | Multi-bucket excavator. | 6-cubic-yard dragline. | Cutter-suction d[redger]. |
|---|---|---|---|
| Weight: tons | 265 | 455 | 850 |
| Horsepower | 475 | 380 | 1,800 |
| Cost, including erection: $ | 170,400 | 204,000 | 258,400 |
| Average output per month: cubic yards | 133,000 | 78,000 | 424,000 |
| Time: Digging: per cent. | 72 | 71 | 77 |
| Greasing: per cent. | 8 | 7 | — |
| Repairs, overhauls, etc., per cent. | 12 | 14 | 8 |
| Weather, and other causes, per cent. | 8 | 8 | 15 |
| Total: per cent. | 100 | 100 | 100 |
| Average costs per month: $ | 2,780 | 2,010 | 5,500 |
| Average unit costs: | | | |
| Fuel: cents per cubic yard | 0·26 | 0·43 | 0·27 |
| Lubricants, etc: cents per cubic yard | 0·30 | 0·37 | 0·30 |
| Cables, spares, repairs: cents per cubic yard | 0·59 | 0·81 | 0·32 |
| Labour on repairs: cents per cubic yard | 0·20 | 0·19 | 0·09 |
| Labour on machine: cents per cubic yard | 0·74 | 0·78 | 0·31 |
| Total | 2·09 | 2·58 | 1·29 |

draglines when those machines were working at the same time and unde[r] similar conditions on the Circulatory canal*. Those costs referred onl[y] to straightforward work, and it had to be borne in mind that for other part[s] of the work which were not straightforward the multi-bucket machine wa[s] unsuitable and only the draglines could be used.

For the excavation of the Axios diversion-channel, with a maximum bed-width of 75 metres, it was decided to use a diesel-electric cutter suction dredger (the water for flotation being obtainable from the existin[g] river), instead of using either draglines or multi-bucket excavators in th[e]

* Journal Inst. C.E., vol. 5 (1936–37), p. 263 (March 1937).

dry. The average operation costs for that dredger were shown also in Table X; and it could be seen that the resulting costs certainly justified the choice of a dredger for that particular work.

**Mr. J. P. Kriel** referred to the location of the dam of the Vaal-Hartz Irrigation Scheme in South Africa (p. 36 §). The Vaal dam, which stored water for the scheme, was actually below the junction of the Vaal and Wilge rivers, south of Johannesburg, whence the water flowed down the Vaal river for abour 300 miles to a weir 40 miles from Kimberley.

For the use of a 1-cubic-yard steam dragline in 1931 in an excavation about 600 feet by 60 feet by 9 feet in connexion with the Rhodesian Railways in Portuguese East Africa, the cost was found to be 7½*d.* per cubic yard, inclusive of depreciation and overhead charges. That figure was kindly supplied by Mr. D. Hole, of the Irrigation Department of the Union of South Africa, and formerly of the Rhodesian Railways.

**Mr. J. P. McNamara** observed that, as a user of draglines, power-shovels, and skimmers, of from ⅜ to 3 cubic yards capacity, working in the stratified deposit shown in *Fig. 15* (facing p. 30 §), his experience was that the modern dragline was the most efficient, owing chiefly to its wide range of operation—an advantage which enabled the attendant transporters necessary when power-shovels and skimmers were used to be dispensed with.

The Author might have dealt more fully with the improved performance which could be obtained from a selected type of bucket, as compared with the only type available up to a few years ago. All other conditions being similar, the replacement of one of the original-type buckets by one of a more suitable type had enabled an increase of nearly 100 per cent. to be obtained in the output of selected material from the face shown in *Fig. 15* (facing p. 30 §), with a decrease of 85 per cent. in the cost of bucket maintenance.

Mr. McNamara had found that it was possible on deep faces to co-ordinate the paying out of the drag- and hoist-ropes so as to place the bucket well beyond the boom-head radius without causing material damage to it. The gain effected by working at that maximum radius, plus the saving in extra capital expenditure in the purchase of a larger machine, in many cases more than offset the drawback of the increased cycle-time, plus any damage to the bucket which might occur when the smaller machine was used.

Increased efficiency would be obtained if it were possible to hoist the loaded bucket on arrival at the boom-head radius. The existing method of balancing the bucket made that impossible. Under certain conditions the bucket could be fully loaded after dragging it less than twice its own length from a point well beyond the boom-head radius, but it was not possible to hoist it so as to prevent spillage, until it had been dragged in to a point

---

§ *Ibid.*

anywhere between two-thirds of that radius and the fairlead. On long-boomed machines considerable time was lost in the operating cycle. The estimated power wasted was more than that required to fill the bucket, and unnecessary wear was caused by what appeared to be a needlessly long drag.

**Mr. H. K. Scott** observed that the Paper was timely, because economy of man-power in handling materials was particularly necessary when an intensive national effort was required. The last war had given an impetus to the use of mechanical plant of the digger type, as exemplified in the ironstone quarries of the Midlands, with the result that a largely increased output was obtained and many men, hitherto employed for getting the mineral by hand, were released for active service. Customs in Great Britain were modified more slowly than in some other countries, notably the United States, and the application of labour-saving equipment for handling materials had lagged behind to the extent of a decade or more. It was true that mechanization tended to create unemployment, but a remedy for that had to be found in some other way than by perpetuating the use of hand-labour for operations involving arduous physical effort, when in other countries, even where the standard of living was low, modern equipment for the handling of materials was in general use, and was obtaining outputs unattainable by hand. Tribute should be paid to the pioneering work carried out in the United States on the design and utilization of excavating equipment, for the efficiency of the machines of to-day was largely due to the field experience in that country, of different types of plant operated under varying conditions.

Since the last war, the internal-combustion engine and the electric drive had largely superseded the steam unit. The figures of comparative costs given in the Paper were, however, probably weighted against coal, and it was not unreasonable to suppose that under war conditions in Great Britain, steam-operated plant might again be found necessary.

The use of the dragline excavator for restoring land in the ironstone area of the Midlands was most effective with overburden up to 10 feet in thickness, and the work could be carried out for a fraction of the cost of the hand-labour employed for that purpose until a few years ago. Where, however, the ironstone cover was much thicker, reaching 50 feet or more, similar treatment of the ridge and furrow, locally known as "hill and dale," of the dump, was not possible, and afforestation appeared to be the only remedy for the surface damage produced by the quarrying operations.

Owing to the desirability of obtaining the maximum dumping radius from any machine, and especially from those of the larger type, it would appear that the use of an alloy steel of high tensile strength and resistance to fatigue and corrosion, would give an appreciable reduction in weight of the boom, without any loss of strength, as compared with the use of ordinary steel.

The costs given in the Paper were useful, but they would have been

more valuable if they had included figures for interest and amortization of capital expenditure. One of the greatest difficulties in excavating plant was to keep it fully employed, especially in the case of the larger units. In practice that was probably never achieved, and when for any reason the efficiency of operation was low, the incidence of capital cost, depreciation, and overhead expenditure would be appreciable.

It might be of interest to mention that the application of the dragline principle was becoming general in underground work for loading mineral at the face or in the stopes into wagons, by means of a bottomless bucket, as indicated by the "Dragveyor." It was increasingly difficult to secure labour willing and able to undertake work involving intense physical effort, especially underground, and the scraper had reduced appreciably that drawback of mining.

**Mr. E. G. Walker** observed that the Paper emphasized, by reference to some of the more striking uses to which dragline excavators had been put, the great advances that had been made in a comparatively short period in that method of excavation. Although the Author referred to the use of draglines for excavation of hard materials and rock, and gave examples, the natural conclusion that was to be drawn from the basic principle of the machine was that it was suitable principally for soft materials, such as those for which it was employed first. By far the greater part of the materials that had to be moved by excavating machinery were of a character which did not require high pressures on the bucket teeth. There was another group, smaller in magnitude, of materials which could be excavated without preliminary blasting, but which, nevertheless, required considerable effort in breaking down. For such materials the power-shovel was bound to be the more suitable machine in many cases, if only by reason of the greater rigidity with which the bucket was held to its work, and, consequently, the greater force which could be exerted by it to break the face. There was, however, no doubt that the ability to excavate below the level of the surface on which it stood gave the dragline a substantial advantage for the removal of broken rock, similar to that referred to on p. 36 § and illustrated in *Fig. 22* (facing p. 35 §).

It would add to the value of the Paper if the Author would give some comparison of performances and costs of operation of draglines and power-shovels for conditions in which the two were comparable.

After the several descriptions of the varied applications of draglines to large and frequent excavation-problems that were given in the Paper, the Author's statement on p. 29 § that the excavation of sand and gravel "is one of the best known applications of a dragline" seemed hardly in accordance with fact. Although the dragline, in common with almost every other type of excavating machinery, was in use for the digging of

§ *Ibid.*

gravel deposits, its use was by no means general. For dealing with under-water deposits, and also for excavating in dry pits, the dragline might be a useful machine if the nature of the deposit and the circumstances of its development were favourable. That was not, however, generally the case.

From consideration of the costs of general excavation which were given in the Paper, it did not seem likely that the dragline could, in general, give such economical results as the hydraulic methods which were in more frequent use in modern wet pits. The hybrid system of dragline and gravel-pump referred to in the Paper might be suited to a special case, but it was difficult to believe that a process of lifting the material by bucket and then dumping it into anything up to 30 feet of water, to be picked up again by a pump, could be suitable for use in the normal circumstances of gravel pits.

A gravel so consolidated that it could not be broken down by a modern gravel-pumping plant was an exceptional material; having regard to the competitive nature of the gravel trade, it was questionable whether such a deposit would be suitable for commercial exploitation. If it were required to excavate such material, it seemed hardly likely that a bucket pulled over the face of the gravel and dependent entirely on its own weight for its bite would do what a powerful gravel-pump could not do. There seemed to be little ground for expecting any future tendency towards the replacement of gravel-pumping plants by draglines.

The Author drew attention to one of the most fruitful sources of waste in gravel-pumping when he referred to the length of the pipe-line. There was no doubt that in many pits there was a tendency to work with pipe-lines that were too long to be efficient. That naturally arose from the steady increase with time of the distance of the excavation face from the washing and grading plant. When the economical limit of direct pumping had been reached, discharge from the gravel-pump into a dumb barge or hopper, which was towed across the pond and discharged at the main plant by a shore pump, was usually the cheapest method. Water-carriage was less costly than that by railway and tip-wagon, and a shore pumping plant usually obviated the necessity for further washing. It was unlikely that a dragline operating in conjunction with barges and grabbing cranes, and presumably requiring also further washing plant, could be operated as economically as a complete hydraulic plant.

The conditions in a dry pit were different. It was quite possible that the dragline might show advantage over other forms of excavator in that case. If data were available, a comparison of draglines with power-shovels, bucket-excavators ("land dredgers"), grabbing cranes, etc., would be of value in that connexion. In gravel workings of magnitude, however, the dry pit was the exception rather than the rule.

The case cited by the Author on p. 29 § of a pit in which a 15-foot

§ *Ibid.*

bed of gravel was covered by 15 feet of clay was also exceptional. Only abnormal circumstances could make the operation of such a deposit a workable proposition commercially.

**The Author,** in reply, observed that there was very little doubt that the dragline excavator was capable of dealing with rock much better than most dragline users realized, and the amount and cost of explosives necessary to blow rock out of a face to enable a dragline of suitable size to deal with it effectively was not excessive. He considered that a maximum of 25 per cent. more explosives might be necessary, as compared with a shovel of similar size, but on most jobs it would be less than that figure. The main point was that the rock had to be broken to a size comparable with the power of the machine and the capacity of the bucket. Referring to *Fig. 2* (facing p. 14 §), the machine illustrated weighed only 40 tons and carried a 1¼-cubic-yard rock-type bucket; yet no trouble was experienced in dealing with the rock, which resembled basalt or granite. The Author had been informed that an output of 90 cubic yards loose measurement per hour was obtained under the conditions illustrated, and that the working cost per cubic yard was much less with the dragline than with a shovel of similar size working in the same cutting, but standing, of course, in the bottom. Admittedly, that was largely due to the material from the shovel having to be transported to the surface by means of wagons, whereas the dragline dumped the material direct. That statement also partially answered Mr. Walker's query. Actual costs of the two types of machines were not available. Another example known to the Author was in ironstone at Frodingham, which was excavated without the least difficulty by a dragline fitted with a 1¾-cubic-yard bucket, using only the same amount of explosives as for a steam shovel, of 2¼-cubic-yard capacity, working under the same conditions.

Drag-rope wear, as referred to by Mr. Bailey, seemed inherent in draglines, in spite of the joint effort of users, wire-rope makers, and excavator-manufacturers to reduce it. It was due to a variety of causes, including bending around the fairlead pulleys and dragging the rope through what was frequently gritty material, to which causes was frequently added the effect of throwing the bucket for dumping, which often resulted in the rope spinning. To increase the effective life of the rope it was advisable to turn the rope end for end when it was about half worn out, or, if the length of rope allowed, to cut off several feet from the "digging" end so as to alter the wearing position.

To reduce overhead costs per cubic yard, mentioned by Mr. Scott, it was advisable in the case of large expensive machines to work them in two, or if possible three, shifts per day.

The Author could assure Mr. Walker that he knew of many places where even a small dragline had successfully excavated gravel too com-

§ *Ibid.*

pact for the gravel-pump to deal with, and he did not agree that the diggin[g] ability of a dragline bucket depended entirely on the weight of the bucket as, although the latest type of dragline bucket was considerably lighte[r] than the old type, it would deal better with much harder and more compac[t] material.

Mr. Fairclough's observations on ultra-light buckets backed up th[e] statement on p. 12 § that "Users are recognizing that the lighter the bucket consistent with its ability to dig, the greater the efficiency or pay load. . . .'. It should be recognized, however, that manufacturers had to supply a bucket "to stand up against, not only the digging stresses, but possibly a certain amount of maltreatment . . .," as mentioned on p. 10 §.

Mr. Fairclough's observations would probably be of interest to Mr Walker, in that they showed that the digging ability of a dragline bucket did not depend entirely on its weight.

§ *Ibid.*

# CORRESPONDENCE

## ON PAPERS PUBLISHED IN APRIL 1940 JOURNAL

### Paper No. 5216.

## "The Sewage-Disposal of Delhi." †

By John Aldhelm Raikes Bromage, M. Inst. C.E.

### Correspondence.

**Mr. A. W. H. Dean** observed that Mr. Townend (p. 189 §) was correct in stating that the final choice of a "Simplex" plant for Delhi was dictated by the ability of the contractor to tender not only for the disposal plant but also for various additional items, such as quarters for the staff, etc., all of which were included in his tender, in which the rate quoted for the job as a whole was lower than that of any competitor.

A year or two before the scheme for the general disposal of sewage in

† Journal Inst. C.E., vol. 14 (1939–40), p. 157 (April 1940).
§ Page numbers so marked refer to the Paper (Footnote (†) above).—Sec. Inst. C.E.

Delhi was finally decided upon, a small plant was installed to serve the Imperial Agricultural Research Institute at New Delhi. That Institute was on the other side of the local main watershed; hence disposal to the existing sewerage system was not easy, and an independent disposal scheme was designed. It also had the advantage that it gave a considerable supply of sludge which could be used as manure, and also of effluent which could be used for irrigation purposes in the Institute. Mr. Dean

ANALYSES OF RAW SEWAGE: PARTS PER 100,000.

| | Free and saline ammonia. | Albuminoid ammonia. | Oxygen-absorption: 3 minutes. | Oxygen-absorption: 4 hours. |
|---|---|---|---|---|
| April 1937 . . . . . . | 12·50 | 5·0 | 4·10 | 11·06 |
| May ,, . . . . . . | 6·25 | 1·79 | 1·90 | 5·03 |
| June ,, . . . . . . | 6·25 | 1·39 | 1·26 | 5·38 |

MONTHLY AVERAGE OF THE EFFLUENT-ANALYSIS BASED UPON WEEKLY ESTIMATIONS: PARTS PER 100,000.

| | Suspended matter. | Free and saline ammonia. | Albuminoid ammonia. | Nitrate. | At 65° (18·3° C.) B.O.D. |
|---|---|---|---|---|---|
| July 1937 . . . . | 0·33 | 0·12 | 0·06 | 1·53 | 0·32 |
| August ,, . . . . | 0·54 | 0·27 | 0·11 | 1·40 | 0·83 |
| September ,, . . . . | 0·31 | 0·08 | 0·05 | 1·50 | 0·41 |
| October ,, . . . . | 0·32 | 0·12 | 0·05 | 1·50 | 0·29 |
| November ,, . . . . | 1·03 | 0·22 | 0·09 | 2·16 | 0·58 |
| December ,, . . . . | 1·55 | 0·29 | 0·14 | 2·34 | 0·76 |
| January 1938 . . . . | 1·68 | 2·45 | 0·14 | 0·35 | 0·84 |
| February ,, . . . . | 1·04 | 0·58 | 0·12 | 0·90 | 1·26 |
| March ,, . . . . | 1·29 | 1·30 | 0·13 | 0·41 | 2·85 |
| April * ,, . . . . | — | 1·16 | 0·14 | 0·51 | — |
| May ,, . . . . | 1·12 | 0·45 | 0·10 | 1·01 | 1·35 |
| June ,, . . . . | 0·43 | 0·30 | 0·07 | 1·15 | 0·70 |

* Average of 14 estimations.

was in charge of the installation of that plant. The activated-sludge plant at the Institute had been working since April 1937, and was dealing with 40,000 to 50,000 gallons of sewage daily from a population of about 1,500 people. About 30,000 to 40,000 gallons of effluent was being obtained daily, and was used for irrigation purposes. The sludge was dried in the sun over drying-beds. Analyses of raw sewage-effluent after purification, and of sludge, were made regularly.

Sewage had been taken into the plant since the last week of January 1937. Systematic analyses of the effluent had been made since April

1937. The staff consisted of one mechanic, who also worked as an engine-driver for one of the shifts, two engine-drivers, three oilers, and four sweepers.

There were two compressed-air engines, which were operated alternately by electricity for the continuous aeration of the tanks. The total power-consumption was about 60 units per day.

The analytical figures for raw sewage and effluent, on p. 553, indicated that sewage-purification by the plant could be taken to a stage of purity of the effluent according to the standards laid down by the Royal Commission on Sewage-Disposal, namely, 3 parts per 100,000 of suspended matter and 2 parts per 100,000 of biochemical oxygen demand.

In the following analyses of the sludge on the dry basis, the results were given as percentages.

| | | March 1937. | May 1938. |
|---|---|---|---|
| | Moisture | 4·64 | 3·11 |
| On dry basis | Loss on ignition | 47·43 | 26·70 |
| On dry basis | Mineral matter | 52·66 | 73·30 |
| On dry basis | Sand and silica | 35·98 | 60·02 |
| On dry basis | Total nitrogen | 4·02 | 2·00 |
| On dry basis | Total $P_2O_5$ | 2·14 | 1·50 |
| On dry basis | Total $K_2O$ | 0·99 | 0·42 |

The 1938 sample revealed a considerable increase in mineral matter and sand, and a corresponding decrease in loss on ignition, total nitrogen, total $P_2O_5$, and total $K_2O$.

With regard to the manurial value of the sludge, no systematic field experiments could yet be arranged at Delhi, as some time would be required for the land to become ready for field experiments. Preliminary experiments had been carried out in an area of 9·47 acres near the sludge-plant, and the yield of oats and barley had been very satisfactory.

**Mr. F. C. Griffin,** who was the Author's successor at the Delhi sewage-disposal works, remarked that a few particulars as to the present operation of the works might be welcomed. The contractors undertook, in their original guarantee, to purify a screened domestic sewage without trade wastes, having an analysis according to an attached schedule. The schedule contained certain analyses made in 1934, and other analyses made in January 1935, and the figures varied widely. For instance, in the 1935 figures, the oxygen absorbed in 4 hours varied from 2·64 to 6·51 parts per 100,000, and the suspended solids from 19·0 to 86·0. The average of the figures was given in the first column of the following Table. The analyses made during the tests in September 1938, and those of the sewage now, were given in columns 2 and 3.

The crude sewage samples were taken at the point where the sewage

| | 1935. | September 1938. | May 1940. |
|---|---|---|---|
| Total solids: parts per 100,000 . . . | 162·36 | 120 to 145 | 120 to 140 |
| Suspended matter: parts per 100,000 . | 56·92 | 50 to 75 | 60 to 70 |
| Oxygen absorbed in 4 hours: parts per 100,000 . . . . . . . . . . | 5·24 | 6·8 to 11 | 8 to 12 |
| Free ammonia: parts per 100,000 . . | 8·63 | — | 3·0 to 3·8 |
| Albuminoid ammonia: parts per 100,000. | 1·12 | — | 0·8 to 1·2 |

entered the purification plant at Okhla, the screening and removal of detritus having been effected 3 miles farther back, at Kilokri.

The analyses of effluent were then as follows:—

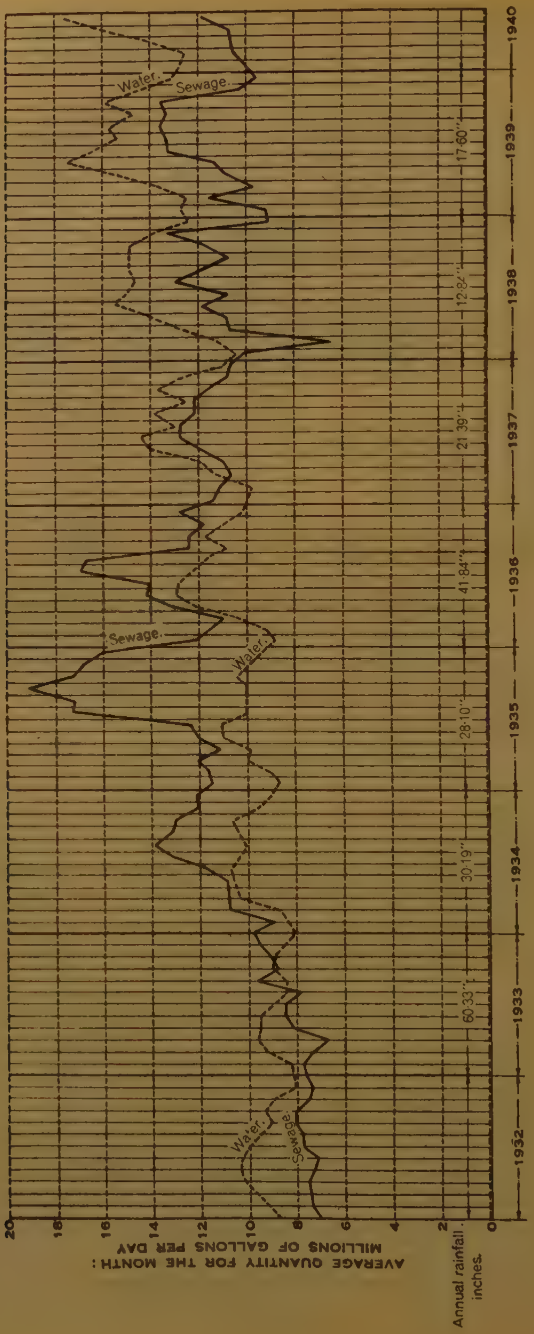
| | | | | |
|---|---|---|---|---|
| Total solids . . . . . . . | From 60 | to 65 | parts per 100,000. |
| Suspended matter . . . . | ,, 1·1 | ,, 1·4 | ,, ,, ,, |
| Oxygen absorbed in 4 hours . | ,, 1·1 | ,, 1·4 | ,, ,, ,, |
| Free ammonia . . . . . . | ,, 1·8 | ,, 2·0 | ,, ,, ,, |
| Albuminoid ammonia . . . | ,, 0·12 | ,, 0·18 | ,, ,, ,, |
| Nitrites . . . . . . . . . | ,, 0·005 | ,, 0·1 | ,, ,, ,, |
| Nitrates . . . . . . . . . | ,, 0·005 | ,, 0·01 | ,, ,, ,, |
| Biochemical oxygen demand . | ,, 0·8 | ,, 1·4 | ,, ,, ,, |

It was still impossible to state the exact population served; 550,000 was still the nearest estimate, of which 74,300 might be taken as the population of New Delhi. In the hot weather those figures were reduced by a few thousands. *Fig. 10*, p. 556, showed the relation between water-supply and sewage-flow from 1932 until the present date. Up till the end of May 1938, all sewage went on to the old Kilokri sewage farm. The new works were brought into operation in June 1938, and the Kilokri farm was shut down from August 1938. Since then the flow of sewage had been about 80 per cent. of the water-supply, and at the end of May 1940 was about 13 million gallons per day. That was about the maximum dry-weather flow. For a population of 500,000, the quantity of sewage per head per day was 26 gallons. That was the hottest season of the year, when the population was slightly less but the water-consumption per head was considerably increased.

*Fig. 11* (*a*), p. 557, showed typical diurnal variations of flow as recorded at Okhla, from a minimum rate of 4 million gallons per day at 6.0 a.m. to a maximum of 19 million gallons per day at 2.0 p.m. The second peak was from 9.0 p.m. to midnight.

*Fig. 11* (*b*) showed the diurnal variations at Kilokri, at which point the peaks occurred 2 hours earlier than at Okhla. That chart was obtained by combining the readings of the two meters on the rising mains, whilst an addition of about 0·5 million gallons per day had to be made to allow for the periods when the main-pumps were standing and the detritus-pumps were running, during which the meter did not record the detritus-pump flow.

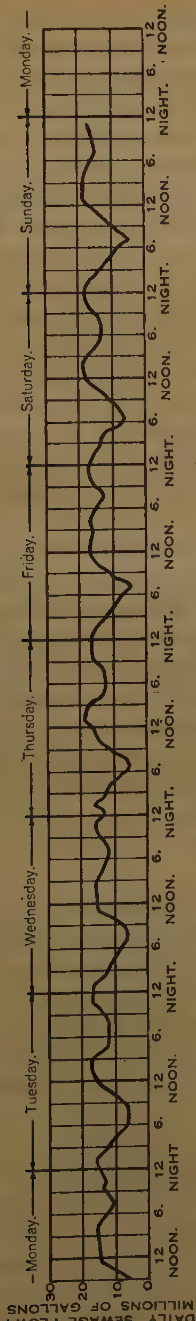
*Fig. 10.*




MONTHLY AVERAGE RATES OF WATER-SUPPLY AND SEWAGE-FLOW.

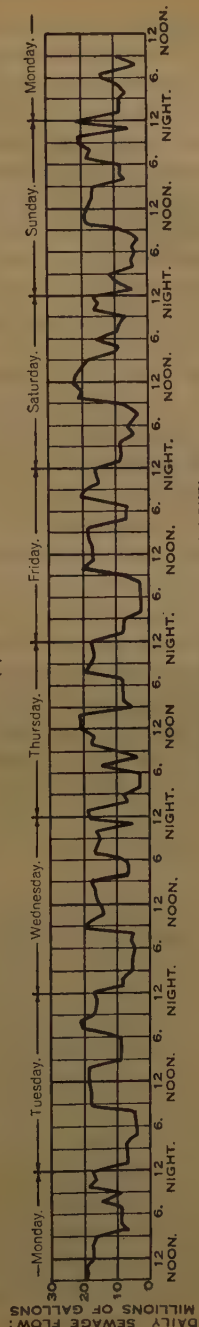
*Figs. 11.*

(*a*)




READING FROM FLUME-RECORDER AT OKHLA.

(*b*)




COMBINED METER-READING AT KILOKRI.

DIURNAL VARIATION OF SEWAGE-FLOW.

In the dry season, the quantity of detritus taken out by the hydrauli separator plant was eight wagons of 18 cubic feet capacity per day. loaded wagon contained about 30 per cent. water, and the daily quantiti of solid dry matter removed was between 3½ and 4 tons. The bulk of tha material was floating, or nearly floating, wood, charcoal, and ashes. A grea deal of silt and fine sand passed the entrances of the detritus plant, ar accumulated in the open channel between those entrances and the suctio pits of the pumps. That had to be cleared every 4 weeks by hand—a operation costing about 50 *rupees* (£3 15*s*.)—and the quantity remove each time was about 200 tons. The removal was effected by hand-stirrin and by pumping silt and water together to low land near the head of tl rising mains.

Some fine sand was also carried through with the sewage to Okhl where, however, it probably assisted quick settlement in the priman settling-tanks. There was no deposit either in the gravity-duct fron Kilokri to Okhla (gradient 1 in 2100) or in the 14-foot well at the hea of the rising mains.

The wells of the detritus plant were 13 feet 6 inches in diameter, wit about 135 square feet net horizontal cross-sectional area. When th sewage-flow was divided, half over the weir and half through the wel giving, say, 11 cusecs through the well, the upward velocity in the we was 0·08 foot per second. The corresponding horizontal velocity of th remainder of the sewage-flow over the weir (4 feet wide and about 1 foo deep) was 3 feet per second. That would mean that the bottom 3 feet c water went into the well, and that the uppermost 1 foot passed over th weir. In the separator, an upward velocity of 0·5 foot per second woul be given when the detritus-pump delivered 4 cusecs vertically down th centre.

The detritus-pump worked intermittently—10 minutes running an 20 minutes idle.

As regards the statement that sludge-digestion was considered to revolutionary for Indian conditions, it might be pointed out that sludge digestion, with collection of sludge-gas for supply to an 80-horse-powe engine, had been in operation at Bhatpara, Bengal, since 1933.

At Delhi, by February 1939 (when Mr. Griffin took over from th Author), the trouble due to flies breeding in the dried or semi-dried sludg from the drying beds was considerable. A month or two later, the troubl had developed into a serious public nuisance, concerning which the medica authorities were very alarmed. The nuisance spread to villages as muc as 4 miles distant. Prompt action was necessary, and within a month temporary digestion-tank was constructed. That was partly for demon stration purposes, and partly for dealing with at least a portion of th sludge. Immediate possession was also taken of low-lying plots of lan nearby, which were formed into lagoons in which the fresh liquid sludg could be digested. Direct attack was also made on the heaps of maggot

infected material by means of flame-throwers and brick furnaces. One large heap was rendered innocuous by throwing an earth-embankment around it and flooding it. The drying-beds were put out of action completely, except for such small quantities of undigested sludge as were required for immediate dispatch from the works to purchasers. In the meantime, preparations had been made for digestion-tanks to deal with the whole flow of sludge, and those were completed in April 1940. There were twelve open tanks, each averaging 210 feet in length by 37½ feet in width, with a 12-foot depth of sludge. The tanks were unlined, though probably brick-pitching at the ends, for 4-foot depth, would be added. The tanks were placed so that the sludge gravitated into them from the main works. There were three draw-off valves at the outlet end, one drawing from the foot of the bank, the second drawing from a point 30 feet out from the bank, and the third drawing from a depth of 4 feet. The total capacity of the tanks was 1,000,000 cubic feet, that was, 2 cubic feet per head. That capacity was found sufficient at the Bhatpara works. The digested sludge discharged by gravity into forty new unlined drying-beds, with a total area of 43,000 square yards. Those were intersected by surface drains and roads, the drains discharging seepage-water or rain-water by gravity into the adjoining main effluent-channel. The roads gave access to each separate bed, so that lorries or carts could be driven right up to the dried sludge, and loaded direct. Overflow-water from the tanks flowed to pumps which could discharge the water either back to the beginning of the plant for re-treatment, or into a lagoon. That lagoon was 2 acres in area, and was located at the top of rising ground from which the effluent-water irrigated about 30 acres of land which would otherwise be out of command. The seepage-water discharged by the drains around the tanks and beds amounted to about 2,000 gallons per hour, even in the dry hot weather. The digestion tanks and new drying-beds cost 60,000 rupees.

Large quantities of digested sludge had already been sold. For production of a better quality of manure, a proposal was under consideration for drying-out surplus activated sludge separately, and mixing it in a grinding process with an equal quantity of digested sludge. Such a mixture was found to be quite safe from fly-breeding, even when it was left lying about in heaps and allowed to become wet again. Samples of that manure had been produced, and were under trial. The analysis was as follows:

| | |
|---|---|
| Nitrogen . . . | 2·275 per cent. |
| Phosphate . . . | 1·402 ,, ,, |
| Organic matter . | 49·2 ,, ,, |

The manure (in powdered form) could be dispatched in bags by rail, or delivered to places near at hand by covered lorries. The railways gave a concession rate for manures at 1 *anna* 5 *pies* per *maund* (2·27*d*. per cwt.) for distances up to 50 miles.

Meanwhile the purified effluent was being used to irrigate about 3,00 acres of land, and results far superior to those obtained on adjoining cana irrigated land were being obtained. By taking off effluent at the highe level possible at the outlet end of the plant, it was found to be just possibl to pass the water by gravity as far as Badarpur. The full value of th nitrogen so supplied could by no means be realized, but rates were bei charged double those for canal water. The medical authorities ha stipulated that no crops which were eaten raw should be grown. The Boar was now laying out a 41-acre experimental agricultural farm. That woul be equipped with a 12-foot well 45 feet deep, electrically pumped, so tha experiments could include irrigation with effluent only, with well-wate only, and with mixtures of the two. On the main area, wells formed th only sources of water for dilution. Water could be obtained from th Agra canal only by pumping, and as the canal authorities of the Unite Provinces had not sufficient water for irrigation in their own region they were not likely to give water to the effluent-irrigated area of Delh province. The canal authorities would allow effluent-water to be passe into the canal only when the dilution was 1 to 250, because the canal-wate was used by villagers for drinking.

Sludge-gas was not being collected at present, because electricity wa supplied at 0·45 *anna* per unit by another public body of the same province A scheme had been suggested, however, with a capital expenditure of th order of £75,000, for collecting the gas and using it in motor-vehicles. Tha scheme offered a big reward, a return at least four times greater than tha obtainable by generating electric power, but it awaited an enterprisin capitalist.

So far there had been no peeling-off of cement-rendering anywher on the disposal works. The bricks were more porous and of roughe texture than English bricks, and perhaps the adhesion was better. A surfaces were treated with silicate of soda, well rubbed in until no mor could be absorbed. The mixture used was the ordinary jelly diluted wit four times its volume of water. It was found that such surfaces wer smooth, that sludge did not readily adhere to them, and that they wer very impervious and hard. Sludge-gases did not readily attack them Those results could be obtained for 6 shillings per 100 square feet (possibl for half that cost), and were well worth the money.

The 5-foot 6-inch outfall-sewers from Point Q (*Fig. 1*, p. 158§) to Kilok had a gradient of 1 in 2750, and a self-cleansing velocity could be obtaine only by using them one at a time. The penstocks at Point Q, therefor were changed over daily, both being left open together only when rai occurred.

The original contract was for a flow of 16 million gallons per day The contractors were asked to accept 18 million gallons per day, and the

§ *Ibid.*

replied that whilst they believed the plant would carry it, they refused to be financially penalized on that quantity. Actually the plant had carried it quite easily. All three sections were run during hours of heavy flow. At times of low flow, certain cones were stopped. The power-consumption remained within 420 units per million gallons per day, as against the 450 units guaranteed. That was ample to ensure an excellent effluent.

As to the 0·75-cubic-foot tank-capacity, the authorities at Delhi wanted a fully-purified effluent. They decided on price, guaranteed performance, and power-consumption under penalty, and had got what they paid for in good measure.

In the Discussion the point had been raised as to why the wet raw sludge had not been put straight on the land. A small quantity had been spread, but it was not satisfactory on account of smell and flies. In a hot country with gentle breezes, such a smell travelled for miles, and so did the flies. That was why digestion had been introduced. The arrangement now made was such that digested sludge could be gravitated or pumped on to land in the neighbourhood, as an alternative to drying in the specially-prepared beds.

**Mr. J. B. L. Meek** observed that very little information was given on the initial basis of the design. It was stated on p. 159 § that owing to the size of the original outfall sewer being inadequate, a nuisance was created at the point Q. Was there any nuisance above that point? On p. 179 § the average rainfall was given as 40 inches per annum, of which 90 per cent., or 36 inches, fell in 3 months. With such a fall in 3 months the intensity of rainfall at times was bound to be high. If Delhi were drained on the combined system, the sizes of some of the sewers above the point Q, as far as could be made out from *Fig. 1* (p. 158 §), did not seem to be sufficient to cope with a high intensity of rainfall. It would be a disaster if, after putting things right at the point Q, trouble and nuisance should still occur above that point. Would the Author give some additional information on that matter? For what population was the scheme designed? What were the figures assumed for the dry-weather flow per head and the intensity of rainfall?

**Mr. J. R. Taylor** remarked that it was not clear from the information given in the Paper whether the sewerage-system of the Old City and the New Capital draining to the point Q was on the combined, the partially-separate, or the separate system. In any case, however, the principle adopted seemed to be that any flows in excess of 3 times the dry-weather flow at, approximately, 25 gallons per head per day, might be discharged untreated into watercourses and thence into the river Jumna. Such a practice was not permissible in Great Britain, where it would normally be necessary to give partial treatment to flows between 3 and 6 times the dry-weather flow; only when the flow exceeded 6 times would discharge

§ *Ibid.*

of untreated liquid into watercourses be allowed. It appeared that at certain times of the year the river dried up ; was it certain that, at such times, the storm overflows would not come into operation ? It was noted that even the effluent from the disposal-works was chlorinated if it were discharged into the river when the latter was dry ; it would seem to be still more necessary to chlorinate the discharges from the stormwater over-flows if there were no dilution when such discharges reached the dried-up river-bed.

It would be of interest to know whether the dry-weather-flow sewage, at 25 gallons per head per day, was weak, medium, or strong. In Great Britain, it would probably be a medium domestic sewage with little, if any, trade-waste effluent.

Could the Author supply results of analysis of averaged samples of raw dry-weather-flow sewage at Delhi ? They would be of interest as indicating not only the dilution when the flow exceeded 3 times dry-weather flow, but also the load on the treatment-works.

The two outfall sewers had a discharging capacity of 72 million gallons per day, which was 3 times the ultimate dry-weather flow (presumably at 25 gallons per hour per day) and about 5 times the dry-weather flow in 1936. Did the flow to the Kilokri pumping station at present ever reach that quantity, and if so, how was the excess over the present pumping capacity of 60 million gallons disposed of ?

Screening of sewage was stated to be arduous owing to large quantities of bulky material entering the sewers. What was the nature of that material, and had it influenced the design of the screens and raking gear ? Were roughing screens provided to prevent large objects from being caught up on the raking gear ?

It would be of interest to have details of the detritus-wells and of the washing tanks, and also results of an analysis of the detritus before and after washing. It was not clear why the principle of washing the detritus had been adopted ; as screenings were buried, it was natural to suppose that detritus could be disposed of in a similar manner without the necessity for washing it.

With so high a vertical velocity as 1 foot per second, a considerable quantity of the lighter and finer detritus materials would not be settled ; charcoal was stated to be the principal constituent of the detritus, and it was doubtful whether that would be settled at such a velocity.

The detention period of 1½ hour's dry-weather flow (p. 172 §) was very short ; so far as could be determined from the information given in the Paper, the vertical velocity through the preliminary settling tanks, when 3 times the dry-weather flow was passing, would be about 34 feet per hour ; that was a very high rate, which would allow hardly any settlement of solids to take place. In fact, it seemed doubtful whether those tanks, in

§ *Ibid.*

the circumstances, served any useful purpose, although he understood that with the "Simplex" aeration system it was usual to adopt that short detention period in England; in America, however, it was more usual to omit the preliminary settling tanks entirely.

The method for relieving pressure behind the pocket walls was a simple solution of a troublesome problem, but it could be adopted only where watertightness was not of great importance and where leakage of sewage into the subsoil was of no account. No doubt the local subsoil water conditions were such as to justify the method of construction adopted. How had the steel troughs been treated to protect the steel from the corrosive action of the sewage? Unless some special precautions had been taken, the troughs might require fairly frequent renewal.

So far as could be ascertained, the sludge-bed area would provide 1 square yard for about fourteen persons on the ultimate population of 550,000; the normal practice in England was to provide 1 square yard for every seven persons, but possibly the climatic conditions justified the smaller area in India.

The conversion of a water-board into a joint water- and sewage-board was of interest. It was ideal that the authority which extracted river water and purified it for domestic use should again purify it after use before discharging it back to the river. It would be still more interesting to know whether the charges for water covered the cost of treating the sewage as well as that of supplying the water, as they should do. It was to be feared that the complication of administrative authorities in Great Britain was so great as to make the formation of a joint water- and sewage-board a difficult matter, but it was certainly worth the consideration of the Ministry of Health in certain cases. For example, would it not be an advantage, when a new joint water-supply authority was set up, if the promoters were made liable for seeing that the water which they supplied was not only rendered satisfactory for domestic use, but also was made innocuous after use?

**The Author,** in further reply to the Discussion and in reply to the Correspondence, observed that the results obtained from the small plant at the Imperial Agricultural Research Institute were of interest. The design and specifications for that plant were prepared by the Author. The reasons for the choice of the "Simplex" plant for the main installation were not entirely financial. As stated by Mr. Dean, the tender for the "Simplex" plant, with all ancillary buildings, was lower than that of any competitor. The guaranteed power-consumption figures for the "Simplex" plant, per million gallons treated, were the lowest and, as stated by Mr. Griffin, that guarantee had been fully met. Moreover, the accessibility and the smallness of the mechanical and electrical units of the "Simplex" plant rendered it the most suitable for a work employing Indian labour. Such economical results could not be expected from a plant of the size of that at the Agricultural Research Institute as from a

large plant, but 60 units of electricity for 45,000 gallons was equivalent to about 1,300 units per millon gallons, which compared unfavourably with 430 units at the main Delhi plant. It would also be noted that the sewage was more dilute at the smaller plant than at the larger.

Although the main storm overflow was at point Q (*Fig. 1*, p. 158 §) there were other overflows at points above point Q. In designing the works described in the Paper, it was realized that sewers above point Q would, in time, have to be duplicated, and the matter was already in hand.

The seasons in India were very regular, and at times when the overflows came into operation, owing to heavy rain, the river would be in flood. The flow at Kilokri up to 1939 had never reached the capacity of the present pumps.

Only one set of screens was provided. Large material, such as vegetable refuse, sugar-cane, wood, and cinders entered the sewers. The detritus was washed to remove, as far as possible, all organic matter and to reduce the volume to be handled.

The basic principle of charging for treating sewage in proportion to the water-supply had been followed, but certain adjustments had been made to cover the somewhat different conditions prevailing in the various administrative areas.

In reply to Mr. Griffin, experience had fully justified the Author's recommendation to adopt sludge digestion from the beginning, and it was his recorded opinion before he left Delhi, that digestion, in one form or another, would be adopted in the very near future.

The charges for effluent used for irrigation were of interest. At the commencement the Author obtained 1·25 times canal rates for effluent, but it was realized that the charge would rise when cultivators appreciated the value of the service. Doubtless that figure would rise further. The experimental farm, 41 acres in extent, would enable more accurate results to be obtained than would have been possible on the original small experimental farm.

The results so far obtained, after nearly 2 years operation, showed that a plant suitable to the Delhi conditions had been installed, although it was to be regretted that the comprehensive scheme, including sludge-digestion as recommended by the Author, was not adopted in the first instance.

---

§ *Ibid.*

Paper No. 5223.

## "The Hydrology of the Yangtze River." †

By Herbert Chatley, D.Sc. (Eng.), M. Inst. C.E.

### Correspondence.

**Mr. E. H. Essex** had prepared, from Plate 2 and Table IV (page 232 §), columns 1 to 10 in Table V (facing p. 566, *post*). He was himself responsible for column 11, from which the figures in columns 12, 14A, and 16 had been derived. The efficiency factor was derived from observations. A value of $E = 1{\cdot}7$ to $1{\cdot}8$ was a safe factor to use in the design of any straight channel in alluvium; it suited the medium flows of almost any straight length of recordings in rivers and canals, but rivers in flood might well rise to $E = 2{\cdot}5$, which was the value indicated by most test-channel gaugings. For the Nile in flood the value was $E = 2$, and in drought $E = 1$; for the Mississippi it was $E = 2{\cdot}5$ in flood and $E = 2$ in drought; those were the values adopted in column 11, and they yielded the slope-values in column 13, which appeared to be in reasonable accord with the limiting values of column 3. The Chezy values in column 12 and the Kutter values in column 14 appeared to be fairly consistent with general observations elsewhere, in spite of the dog-tooth bottom shown on the longitudinal section.

The regime values for velocity suggested that the river had a tendency to silt up even in flood; but those values were derived from $V = 10{\cdot}8\sqrt[3]{R^2S}$ or $V^3/R^2 = 1{,}250.S$, which was a dimensional expression made equal to a non-dimensional one, so that considerable error would arise beyond certain restricted limits: that was why Dr. Engel * had failed to obtain hydraulic similarity for velocities in Kramer's experimental channel by the use of $R^2S$. In that channel $E = 2{\cdot}5$ to $3$, and $E \,.\log \frac{\rho}{\mu}RV$ gave values for $\frac{C}{\sqrt{2g}}$. It was useful to note that $\frac{2g}{C^2}$ represented, approximately, unit drag per unit area per unit time, in lb. per square foot of wetted surface-area per second. Multiplied by the cube of the velocity in feet per second, it gave the actual drag, in lb. per square foot per second.

**The Author,** in reply, observed that the computations that Mr. Essex had made were very interesting. The Author had made numerous

† Journal Inst. C.E., vol. 14 (1939–40), p. 227 (April, 1940).

§ Page numbers so marked refer to the Paper (Footnote (†) above).—Sec. Inst. C.E.

* Discussion on "Uniform Flow in Alluvial Rivers and Canals." Minutes of Proceedings Inst. C.E., vol 237 (1933–4, Part 1), p. 488.

attempts to compare the Yangtze velocities with formulas ever since the first measurements were made at Wuhu in 1916, but had obtained no conclusive result. Further similar studies had been made by Colonel Stroebe and also by Mr. Hsü Kai, former Engineer-in-Chief to the Hwai River Conservancy Commission. Exponential formulas were certainly applicable, and the Kutter coefficient was about 0·025, but the difficulty of measuring the slope exactly had always made it impossible to arrive at satisfactory conclusions. The Lacey regime condition applied in a general sense, but whether it should be related to average high stage, mean stage, or perhaps even to maximum stage, would depend on the time required to establish bed-regime. Mr. Essex's conclusion, that the river had a tendency to silt up even in flood, might be true to a small extent, but it was difficult to know how to distinguish foreshore- and embayment-silting from bed-silting, and at the present time the Author regarded that as an open question. Lacey's theory was not necessarily supposed to apply to cohesive materials, and the Yangtze bed-material was cohesive throughout the alluvial plain.

---

# CORRESPONDENCE

## ON PAPERS PUBLISHED IN

## JUNE 1940 JOURNAL.

---

### Road Paper No. 2.

## The Engineer's Part in the Promotion of Road Safety." †

By Frank Alan Rayfield, Assoc. M. Inst. C.E.

### Correspondence.

**Professor R. G. C. Batson** pointed out that Memorandum No. 483 (Roads) of the Ministry of Transport on the layout and construction of roads, published in 1937, stated that " In so far as existing road conditions are a contributory factor in the causation of accidents, their improvements demand a close study of the incidence of accidents. It is therefore recommended that map records of accidents should be prepared and maintained by the Highway Authority in co-operation with the Police." It was further recommended that maps should be examined and analysed.

† Journal Inst. C.E., vol. 14 (1939–40), p. 267 (June 1940).

Table V.—Chezy $C$ Values, Kutter $N$ Values, and Lacey's Regime Velocities.

| (1) Station. | (2) Distance downstream from Chenlingki: kilometres. | (3) $1,000\,S = 1,000\,\frac{H}{L}$ | (4) Flow $Q$: cubic metres per second. | (5) Area $A$: square metres. | (6) Mean velocity $Q/A$: metres per second. | (7) Surface-breadth $B$: metres. | (8) Mean depth $D = A/B$: metres. | (9) $R = \frac{A}{B + 2D}$ metres. | (10) $\text{Log}\,\frac{\rho}{\mu} RV$ $(\log \frac{\rho}{\mu} = 5{\cdot}94)$ | (11) $\frac{C}{\sqrt{2g}} = E \log \frac{\rho}{\mu} RV$ $\left(\begin{array}{c} E = 2{\cdot}5 \text{ for H.W.} \\ E = 2{\cdot}0 \text{ for L.W.} \end{array}\right)$ | (12) $C$. | (13) $1,000\,S = \frac{1,000\,V^3}{C^3 R}$ | (14) Kutter's $N$. | (14A) Manning's $N = \frac{\sqrt[6]{R}}{C}$ | (15) $\frac{R}{V}$ | (16) $R^2S$ | (17) Regime $V$.* $V = 10{\cdot}8\sqrt[3]{R^2 S}$ metres per second. | (18) Regime $S$ and $C$.* $1,000\,S = \frac{V^3}{1{\cdot}25 R^2}$ | (19) $C = \sqrt{\frac{1,250 R}{V}}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Chenlingki . . . | 0 | 0·08 | 54,850 | 20,000 | 2·74 | 1,000 | 20·0 | 19·3 | 7·66 | 19·15 | 85 | 0·054 | 0·018 | 0·0193 | 7·0 | 0·020 | 2·9 | 0·044 | 93·5 |
| | 20 | | H.W. | 45,000 | 1·22 | 3,000 | 15·0 | 14·8 | 7·20 | 18·0 | 80 | 0·016 | 0·028 | 0·0196 | 8·6 | 0·0035 | 1·6 | 0·013 | 103 |
| | 0 | 0·06 | 6,530 | 4,000 | 1·63 | 900 | 4·45 | 4·4 | 6·80 | 13·6 | 60 | 0·17 | 0·022 | 0·0213 | 2·7 | 0·0034 | 1·6 | 0·18 | 58 |
| | 20 | | L.W. | 10,000 | 0·65 | 1,900 | 5·3 | 5·23 | 6·47 | 12·94 | 57 | 0·025 | 0·030 | 0·0197 | 8·05 | 0·0065 | 0·92 | 0·003 | 100 |
| Hankow . . . | 230 | 0·022 | 60,750 | 34,000 | 1·80 | 1,900 | 17·8 | 17·6 | 7·44 | 18·6 | 82 | 0·028 | 0·025 | 0·0197 | 9·8 | 0·0087 | 2·2 | 0·015 | 110 |
| | 240 | | H.W. | 48,000 | 1·26 | 3,500 | 13·8 | 13·6 | 7·17 | 17·92 | 79 | 0·020 | 0·027 | 0·0197 | 10·4 | 0·0037 | 1·65 | 0·009 | 113 |
| | 230 | 0·022 | 5,280 | 10,000 | 0·52 | 1,400 | 7·15 | 7·1 | 6·50 | 13·0 | 57·5 | 0·012 | 0·040 | 0·0242 | 13·6 | 0·00057 | 0·88 | 0·0022 | 130 |
| | 240 | | L.W. | 8,000 | 0·65 | 900 | 8·88 | 8·72 | 6·69 | 13·38 | 59 | 0·014 | 0·039 | 0·0242 | 13·4 | 0·0011 | 1·10 | 0·0029 | 129 |
| Kiukang . . . | 475 | 0·02 | 64,350 | 40,000 | 1·60 | 2,500 | 16·1 | 15·8 | 7·34 | 18·35 | 81 | 0·025 | 0·025 | 0·0190 | 9·8 | 0·0063 | 1·95 | 0·013 | 110 |
| | 500 | | H.W. | 50,000 | 1·29 | 5,000 | 10·0 | 9·95 | 7·05 | 17·62 | 78 | 0·027 | 0·023 | 0·0190 | 7·7 | 0·00265 | 1·50 | 0·017 | 98 |
| | 475 | 0·02 | 4,818 | 10,000 | 0·48 | 900 | 11·2 | 10·85 | 6·66 | 13·32 | 59 | 0·006 | 0·047 | 0·0252 | 22·5 | 0·0007 | 0·95 | 0·00075 | 167 |
| | 500 | | L.W. | 10,000 | 0·48 | 1,500 | 6·67 | 6·6 | 6·44 | 12·88 | 57 | 0·011 | 0·036 | 0·0240 | 13·8 | 0·00045 | 0·83 | 0·002 | 131 |
| Tatung . . . . | 705 | 0·035 | 67,670 | 42,000 | 1·61 | 3,500 | 12·0 | 11·9 | 7·22 | 18·05 | 80 | 0·034 | 0·022 | 0·0190 | 7·4 | 0·0049 | 1·8 | 0·023 | 96 |
| | 720 | | H.W. | 60,000 | 1·13 | 4,000 | 15·0 | 14·9 | 7·17 | 17·92 | 79·5 | 0·014 | 0·030 | 0·0197 | 13·2 | 0·0031 | 1·55 | 0·0052 | 128 |
| | 705 | 0·175 | 7,721 | 21,000 | 0·37 | 1,600 | 13·2 | 13·0 | 6·62 | 13·24 | 58·5 | 0·003 | 0·060 | 0·0264 | 38 | 0·0005 | 0·86 | 0·0002 | 218 |
| | 720 | | L.W. | 15,500 | 0·50 | 2,000 | 7·75 | 7·68 | 6·52 | 13·04 | 57·5 | 0·010 | 0·040 | 0·0245 | 15·4 | 0·0006 | 0·90 | 0·0017 | 138 |
| Wuhu . . . . | 825 | 0·0166 | 61,500 | 32,000 | 1·92 | 1,400 | 22·8 | 22·2 | 7·57 | 18·92 | 83·5 | 0·024 | 0·027 | 0·0203 | 11·6 | 0·012 | 2·4 | 0·0114 | 120 |
| | 870 | | H.W. | 53,000 | 1·16 | 4,800 | 11·05 | 11·0 | 7·04 | 17·60 | 78 | 0·020 | 0·025 | 0·0161 | 9·5 | 0·0024 | 1·42 | 0·0102 | 109 |
| Slightly tidal . | 825 | 0·0125 | 7,260 | 20,000 | 0·36 | 1,200 | 16·7 | 16·5 | 6·71 | 13·42 | 60 | 0·0022 | 0·063 | 0·0266 | 46 | 0·0006 | 0·90 | 0·000136 | 240 |
| | 870 | | L.W. | 23,000 | 0·315 | 1,700 | 13·5 | 13·3 | 6·56 | 13·12 | 58 | 0·0022 | 0·060 | 0·0266 | 42 | 0·0004 | 0·79 | 0·000142 | 228 |

* Gerald Lacey, "Uniform Flow in Alluvial Rivers and Canals." Minutes of Proceedings Inst. C.E., vol. 237 (1933–4, Part 1), p. 428.

On page 6 of the Report of the Road Research Board for the period ended the 31st March, 1935, a programme of research was outlined. It was stated that " the problems to be faced in road research can be grouped under three headings :—

" ($\alpha$) Economy in road making and maintenance.
" ($\beta$) Reduction of accident ratios.
" ($\phi$) Solution of urgent practical problems.

" The research programme proper must be based on ($\alpha$) and ($\beta$) ", and on page 22 of the Report it was stated " The Ministry of Transport are making some investigation into the occurrence and causes of accidents."

In view of those definite statements, it was rather surprising that so little had been published on the subject by official sources, and that the Author had found as the principal conclusion of his investigation that " there is obviously wide scope for further investigation into the causes of accidents, and for detailed analysis of existing statistics."

The Author asked why Great Britain had not followed the example of Germany, Holland, Belgium, and France in the construction of motor-roads. It had been suggested that the real answer to that question was that an improvement in the road system of Great Britain would cause a decrease in railway traffic. If the scheme prepared by the Lancashire County Council in 1937 had not been held up by the Ministry of Transport, at least one example of a motor-road would have existed in Great Britain at the present time. The Author outlined suggested disadvantages in the provision of motor-roads advanced by their opponents, and at the same time suggested a suitable reply.

In an Address before the North-Western Local Association of The Institution in October 1935, Mr. G. E. Ashforth, M. Inst. C.E., stated that " there is good reason in a line of policy directed towards the reduction of motor traffic on our public highways by the provision of special motor-ways intended solely for use by motors."

In a Paper published in 1937 *, Professor Batson gave a reason for the construction of motor-roads not included in the four suggested by the Author. In the words of that Paper,

> " Transport is as vital in war as in peace and an adequate road system is essential for defence. The destruction of a main railway line would be a serious handicap to the mobilisation and distribution of troops and materials, but the damage of a main road, although unfortunate, would not be vital in itself. The provision of a system of motor roads can therefore be considered as an essential part of any rearmament scheme. In my opinion, a system of such highways is bound to be provided sooner or later and the longer it is delayed the greater will be the cost."

---

* " Some Problems in Connexion with Modern Roads." *Chemistry and Industry*, vol. 56 (1937), p. 1065.

It was an interesting fact that most of the large schemes of new road-construction had been carried out in Great Britain because of military requirements. Examples were the Roman roads in the years A.D. 100–400, the Cambridge–Isle of Ely road in 1071, General Wade's roads in Scotland in 1726–1737, and the Newcastle–Carlisle road in 1751.

**Mr. Fred Lavis** remarked that the question of "building safety into highways" had received a great deal of attention in the United States and there, as everywhere, a considerable difference of opinion existed as to whether accidents were due to the condition of the road or to faults of the drivers.

Mr. Lavis maintained that, except in a very few special cases, the fault in highway accidents was almost entirely that of the driver, and in many—perhaps most—cases was due to temporary aberration, excessive speed, or defects in the vehicle itself.

It was acknowledged that practically every kind of motor-vehicle could be driven with safety over almost any kind of road, in fact over much territory where no roads worthy of the name existed. It was largely a matter of the rate of motion at which the vehicle was driven, skill in driving, and the condition of the vehicle.

General economic conditions, however, demanded that, in developed countries, roads should be provided over which motor-vehicles could move at reasonable speeds without incurring undue risks either to themselves or to other users of the road, and that meant the provision of smooth, safe and sufficiently wide roads with fair alignment and gradients.

Inasmuch as engineers knew how to build first-class roads and were familiar with the methods and design of the best types of highways, it followed that the problem was one to be solved by judgment rather than by engineering formulas, and such judgment involved consideration of financial and economic conditions, and a balance between the reasonable needs of the drivers or owners of motor-vehicles to move freely and even rapidly and the costs to the general public of providing such facilities.

In the United States it had been determined that for ordinary traffic on main-line highways a two-lane roadway was required, not less than 20 feet wide, fairly smooth, and of substantial construction, with proper shoulders and reasonably long sight-distances. There were, of course, also the three-lane and four-lane highways and the so-called super-highways, freeways, and parkways, on many of which crossings of other highways at road-level had been eliminated. Four-lane highways were now divided by a median strip. Details of such constructions were well known, so that it was a question not of lack of knowledge of engineering design, but of expediency and the necessary expenditure.

Perhaps one of the most valuable aids to safety in the use of the highways was the plentiful provision of easily read signs indicating approach to changing conditions, curves to the left or right, changes of surface, steep gradients, crossings of other roads, approaches to built-up sections, etc.

There was some danger in attempting to draw conclusions or inferences as to highway accidents from such statistics as those presented by the Author in Table I, p. 269 §. In the United States there were densely-populated areas in the east, less densely-populated areas in the south and middle west, and large areas of comparatively small population and heavily accidented terrain in the west. To collect statistics for those regions all together and to compare the whole with Great Britain, Belgium or other countries hardly appeared to permit useful inferences or to have much bearing on the question being considered. Even the comparison of accident-occurrences in the United States with those of Canada was of little value, as conditions in the two countries were very dissimilar.

Perhaps a partial answer to the Author's question (6), p. 269 §, might be that on roads used by light traffic, higher speeds were attained and maintained than in sections where heavy traffic prevailed.

There could be no valid difference of opinion with the Author as to the " heavy responsibility which rests not only upon all road users, but also upon designers of vehicles and roads, to use every means in their power to reduce this heavy toll of life, injury and financial loss " due to highway accidents.

The engineers of the United States charged with highway design, construction, and maintenance, as well as civil road-authorities, took their duties in that respect very seriously, and were continually alert not only to " build safety into highways " but also to maintain the highways in a safe condition, and especially to provide warning-signs of changing conditions or places where caution was necessary.

Mr. Lavis was convinced, however, that the education of the driving public in safe and careful driving, the strict enforcement of traffic rules and regulations, and the elimination of incompetent drivers were the most important factors in highway accident prevention. That matter had been discussed at length and in considerable detail in a Paper * presented by Mr. Lavis at the annual meeting of the Highway Research Board at Washington, D.C., in 1937.

**Professor F. G. Royal-Dawson** remarked that, whilst the Paper contained much valuable information, he agreed with the Author that there was wide scope for further investigation into the causes of accidents. Every situation had its own problem. A whole-time body of specialists, devoted to a study of the fundamental principles of road safety in all its aspects, and the application of those principles to given situations, was needed. The present statistical classification of accidents into supposed causes was unscientific. Many assigned causes might be only contributory factors, or might be entirely irrelevant. It required a specially-trained

§ *Ibid.*

* " Safety and Speeds as affecting Highway Design," Proceedings of the Seventeenth Annual Meeting, Highway Research Board (parts I and II, 1937), Washington, D.C.

mind to search for, or to select, relevant factors, and to interpret then intelligently.

In the case of railways, the Ministry of Transport had its own staff o inspectors who were technical experts, and whose most important dut was to investigate the causes of railway accidents. Their findings wer therefore authoritative. On Indian railways, the responsibilities o Government inspectors were even wider. They had the power to impos speed-restrictions where necessary, and to veto proposals that appeared t conflict with principles of public safety.

On railways, the causes of accidents were of certain well-defined type: For instance, the factor of steering was entirely eliminated, and the drive was not even answerable for running over points which might have bee wrongly set, his responsibilities being confined to the observance of speed restrictions and obedience to signals. On the other hand, on roads, a vas number of heterogeneous factors had to be taken into account. Ther was thus all the more necessity for a body of specialists capable not onl of diagnosing with scientific certainty the cause of an accident, but also o detecting situations which were potentially dangerous. Such specialist should have not only observant eyes and an insight into the psychology o all classes of road users, but also a thorough scientific and practica knowledge of dynamics in relation to the movements of vehicles, and th reactions of such movements to road surfaces, especially on curves, for i had to be borne in mind that the track described by a vehicle prior to a accident was automatically determined by dynamic conditions, whic could be reconstructed by inference from the detailed evidence of the trac itself when visible traces of it remained. The average civil engineer ha neither time nor opportunity for dynamic experiments. Such experiment should be carried out under the ægis of, say, the Road Research Depar ment, or a similar organization. Whilst useful dynamic experience coul thus be obtained, the best ultimate field of research for road psycholog and dynamics combined would be the roads themselves, under all conditio of traffic and on all classes of road. Thus a type of specialist would b evolved, who would form the nucleus of a body to be attached to th Ministry of Transport as advisers on all questions affecting public safet in road-transport. They should have nothing to do with the financi aspect of road administration. All projects and proposed reconstruction however, should have their approval, or should conform with standar authorized by them in the interests of public safety, before constructio was sanctioned. They should also have the right to attend inquiries o road accidents and to give their opinions when deemed necessary, an should from time to time make inspections of road conditions on behalf all road-users. At present, there was no responsible authority to whom t general public could turn for an expert opinion in such matters. Once t need for such an authoritative body was recognized, no doubt suitab personnel could be found.

**Mr. A. J. H. Clayton**, in replying for the Author, observed that a contribution to the discussion from the United States was very welcome, especially in view of the considerable reduction in the accident-rate which had been achieved in that country during the past few years. It was difficult, however, to agree entirely with the second paragraph of Mr. Lavis's observations which, taken by itself, placed the blame almost entirely upon the driver. Mr. Clayton preferred to say that the fault of the driver lay in doing the wrong thing in view of the existing conditions; but the engineer was largely responsible for those conditions, and he could improve them if the necessary money were provided. That was illustrated by the substantial reduction in the accident-rate which had been achieved when suitable engineering works had been carried out. It was true that some accidents would still occur on the perfect road; but even if drivers were ideal, they could not always avoid accidents on badly-designed roads, nor if their vehicles failed them. Therefore, the road engineer, together with those who were responsible for the financial policy, must surely share the blame with the driver.

Engineers knew how to build first-class roads, and so far as new roads and major improvements were concerned the matter was largely one of judgment and finance. There was, however, still a great deal to be learnt regarding the smaller and comparatively inexpensive improvements which the engineer could make with very beneficial results.

It would undoubtedly be interesting to compare the accident-rates of different parts of the United States with those of European countries, as aggregates for widely different areas were misleading. If the accident-rates in the various parts of the United Kingdom were examined, it would be found that although there was considerable variation (due, no doubt largely to chance), there was a definite tendency for the rate of accidents involving personal injury to be lower in the thinly-populated areas than in the towns, particularly the larger towns. In 1936 the rate was 403 accidents per hundred thousand population for England and Wales, excluding Greater London, but was 603 for the Greater London area and 471 for the next three largest cities.

The Author would no doubt agree with the suggestions put forward regarding the necessity for proper scientific study of the accident problem.

The fact that so few accidents were due to failures of the vehicles themselves was probably due in no small degree to the careful investigation made by the Ministry of Transport engineers in conjunction with the manufacturers, when any such accidents did occur. The investigation of other accidents was a more complicated matter, and was, no doubt, one which should be entrusted to engineers who were able to specialize in the study of the problem.

---

Paper No. 5222.

## "Remodelling of the Assiut Barrage, Egypt."†

By John Edward Bostock, O.B.E., M. Inst. C.E.

### Correspondence.

**Sir Bernard Darley** remarked that it would be interesting if th
Author would supplement his very interesting and instructive Paper wit
an explanation of how the length of the impermeable floor, the depth o
the new sheet-piling, and the height of the baffle and deflecting-wal
downstream were determined when preparing the design. So far a
could be ascertained from the Paper, the new floor with the additiona
piling had been designed to give a hydraulic gradient of 1 in 20, based o
the old "creep" theory. In northern India, after several works had failed
exact measurements were made of pressures under existing weirs. Those
and also model-experiments, made it evident that that old theory woul
have to be discarded, and that new methods of design would have to b
adopted. The results obtained from those measurements and experimen
had been fully recorded, and a new theory for the design of such works ha
been proposed*.

The design of the Assiut barrage as now remodelled appeared to b
very safe, if perhaps somewhat extravagant. Sir Bernard Darley woul
like to ask the Author whether any reason existed for such a thick upstrea
floor where the upward pressure was counterbalanced by the weight o
the water above. Normally, half the thickness shown in the diagram woul
have been sufficient.

**Mr. I. W. G. Freeman** wished to elaborate some of the remarks mad
by the Author, with more particular reference to details of constructio
Undoubtedly matters of prime importance in the construction of wo
of the nature described were an effective cofferdam and equally effecti
drainage. On those preliminary works depended the success or failure
any programme of construction, and they were so much dependent one o
the other, that they could well be treated together. Practical consider
tions had shown that, however much might be done in calculating theoretic
hydraulic gradients and in estimating velocities of flow of water throu
sand, such information, although providing a necessary guide, was usele
without proper regard to site conditions and natural phenomena. Su
failures at the initial construction of the Assiut barrage and at the Na
Hamadi barrage had made that abundantly clear, and had, incidental

† Journal Inst. C.E., vol. 14 (1939–40), p. 301 (June 1940).

* A. N. Khosla, N. K. Bose and E. M. Taylor, "Design of Weirs on Permea
Foundations." Publication No. 12, Central Board of Irrigation, India. Simla, 19

shown the advisability of taking every precaution in sudd construction by allowing for any preconceived eventuality and by regarding the additional cost as having been well spent as an assurance for the work as a whole. The tremendous sudd work for the remodelling of the Assiut barrage had been primarily justified by its vital importance, as referred to by the Author, but practical considerations showed that its proportions were not, in general, excessive, and although there were sections of sudd which it seemed could safely have been reduced in size, there were also sections where it was considered advisable to increase the deposit of sand outside the piling to ensure safety.

Wide variations in conditions along the sudd were very well illustrated by the Author in the results of experiments made, which determined the levels of saturation in the banks. However, beyond indicating those variations, Mr. Freeman did not think that the results of experiments given had any reliable bearing on the natural gradient of flow through sand, for the angle of saturation was determined mainly by drainage, and might be as steep in coarse material as in materials having a high degree of impermeability. A sudd formed of coarse material might reasonably be safer than one made with very fine material having a greater resistance to water-flow, for excessively fine material was more easily carried in suspension, and, after saturation, the least flow or pressure reduced it to an unstable condition liable to slump. The fine sand and silt upstream of the barrage which resisted water-flow was for the same reason difficult to drain, had a high degree of capillary attraction, readily transmitted pressures, and had little resistance to the formation of springs or to their movement towards their source, with consequent danger to the sudd. For that reason he believed that the inner row of piling was justified far more as a protection to the toe of the sudd than as a means of lowering the line of saturation. In point of fact, the inner piling actually raised the line of saturation in the sudd, and he believed that an improvement would have been effected had it been driven purely as a protective toe and less efficiently as a cut-off. The raising of the saturation-level in the sudd-bank, whilst probably reducing the velocity of flow, as the Author remarked, at the same time increased the pressure of water being relieved inside the piling and over the working area, and also resulted in "backed-up" water escaping around the ends of the piling where they met the permanent work. It also decreased the general stability of the sudd, and created conditions between the two rows of piling which had to be met by additional sand filling to prevent the formation of an unprotected toe at saturation-level, or by reducing the cut-off efficiency of the inner piling by making holes for drainage.

It was of interest to note that, in spite of the very much greater head on the sudds used for the Assiut barrage remodelling, the quantity of drainage water pumped per hour was almost the same as at the Nag-Hamadi barrage. From records in Mr. Freeman's possession, pumping

on the upstream side of the barrage was only 5 per cent. more than on th
downstream side, in spite of the great difference in heads, and that migh
be accounted for by coarser materials, including the presence of rubbl
providing a freer flow through the sudd on the downstream side. At th
completion of the season's work, and with the inside water at the apron
level of 43·25 metres, the pumping was reduced to 40 per cent. of th
maximum volume pumped at foundation-level, and the pumping on th
downstream side then exceeded that on the upstream by 30 per cen
The method of preparing sumps inside the sudded area before de-waterin
as described by the Author, proved very successful, and, although initiall
expensive, undoubtedly made efficient working inside the sudd possible
week earlier than would have been the case had sumps to be constructe
after de-watering.

Two points in connexion with the design of the work had greatl
impressed Mr. Freeman during construction. The first was the unquestion
able improvement effected by the increased use of pre-cast units, blockwo
in the downstream toe and reinforced-concrete sheet-piling in the cut-o
replacing the type of construction used in the Nag-Hamadi barrag
where concrete had been deposited in water between rows of steel shee
piling. Concreting in narrow trenches excavated below the gener
foundation-level was generally most unsatisfactory, and the difficulti
of dealing with the water were practically insurmountable. If it we
allowed to rise in the trench, the water would invariably reach a heig
above the level of the drained area outside the piling, and would dischar
over the tops of the piling. Pumping in those trenches naturally increas
the volume of water finding its way up through the bottom of the trenc
and induced the formation of springs, which would wash the cement o
of the concrete. The formation of sumps and under-drainage in the tren
was made difficult and almost impractical by the quantity of sand whi
rose quickly from the foundation. Apart from the very desirable quali
of flexibility imparted by pre-cast work, it ensured the use of good materi
and, as the Author pointed out, it involved far less work in the rive
bottom. It would seem, therefore, that the extension of that princip
to the upstream toe would have had much to commend it.

The second point was the use of hand-packed dry rubble underlyi
the blockwork aprons. It occurred to Mr. Freeman that a correspondin
thick layer of gravel would be preferable, and its substitution for lim
stone rubble would undoubtedly effect a substantial economy on th
item. The placing of rubble on which to lay blocks true to line and le
was skilled masons' work, and such care had to be taken to provide
satisfactory surface to receive the blocks, that, with the large faces of t
stones upwards, it was difficult to ensure the proper filling of the voi
However much care might be taken in hand-packing comparatively la
and rough rubble, it was practically impossible to obtain a perfectly le
surface which would give an even bearing for the superimposed blo

Furthermore, during the time elapsing between the placing of the rubble and the laying of the blocks set on it, some settlement might take place due to the flow of the water over the area, and damage to the pitching might be done by traffic, the result being that packing under the blocks, although avoided as far as possible, became to some degree necessary. Gravel was plentiful and obtainable cheaply in most places in Egypt, and it was easily handled and could be laid and screeded to correct levels by unskilled labour. It was well graded and had a small percentage of voids; it would consequently be less inclined to settlement, and would form a very much better filter than rubble. Gravel, being of flint, resisted erosion much better than limestone. It was of interest to note that, in spite of the tremendous erosive action which had taken place on the downstream side of the barrage, gravel which had been deposited at the initial construction of the barrage was still present, and, as mentioned by the Author, formed an excellent foundation for the new work. Quantities of limestone rubble, tipped for protection of the barrage during recent years, were found to have suffered considerably from erosion, most of it being rounded and small, whilst there was evidence that much had been completely destroyed.

**Mr. H. F. Wilmot** observed that the key to the success achieved in the work lay mainly in two factors; namely (1) the design of the sudds (" sudd " or " sadd " was the Arabic word for a barrier or obstruction), and (2) the new process of cementation with bituminous emulsion. Once the danger of percolation of water under the floor, or excessive uplift in the enclosed areas, was provided against, the work of reconstruction could be proceeded with as rapidly as organization permitted.

Since the barrage was commenced, the technique in grouting had been revolutionized, so far as cement was concerned, by using the cement mixed with sand (optional) and water to form a colloidal mixture. The effect of that was to enable the grout to penetrate as a liquid, and, therefore, grouting of sand in situ, impossible with neat cement mixed with water, became practicable. It was quite possible that that process might have been used equally successfully instead of the bitumen emulsion for the cut-offs. In the employment of steel sheet-piling, which could be driven very rapidly into the bed of the Nile, the modern engineer had a method of assisting construction that made him forget the enormous simplification it had effected on problems which the engineer of a previous generation had no easy or short way of overcoming.

With regard to the design of the new work, it had already been stated that the old barrage regularly held up a head of water greater than the 2 metres for which it was originally designed. It should be emphasized that the strengthened structure could now hold up 4 metres of flood-waters. Comparisons between the new and the old work could be somewhat misleading unless that was kept in mind.

It was mentioned that it was decided not to build a completely new

barrage, and it was inferred that the reconstruction of the old barrage w a more venturesome project. Actually, a change in site would ha deprived the contractors of the benefit of being able to regulate on t existing gates in low water, whilst a site immediately downstream was n practicable owing to the existence of a large scoured depression. A com pletely new site involved finding a straight reach in the river of suitab length and suitable location for the take-off of the head-canals.

It seemed a pity that the impermeable clay underneath the rubb apron upstream had had to be removed when it was found to be in su excellent condition, but that was presumably essential in order to achie the required thickness of floor. The gravel foundation underneath th downstream portion of the floor, although excellent as a base for concret was surely an undesirable element in so far as it tended to encoura "creep" or percolation under the floor; doubtless the efficient groutin of the whole floor after its completion would reduce that tendency. Pr sumably the removal of the gravel involved extra work, which, it w considered, was not worth while.

The conditions commented on above were undoubtedly the chi reasons that infiltration during construction was greater downstream tha upstream. An additional reason was that, under normal working co ditions for the barrage, there was no uplift upstream, and, therefore, th seepage which took place tended to cause a denser material to remai under the floor upstream. Confirmation of that was surely to be found i the smaller amount of grout required under the floor upstream.

The use of monolithic concrete to form the solid apron or floor bot upstream and downstream of the structure had, generally speaking, n found favour in the past for that type of work, the reason being that rubb masonry was more accommodating in its behaviour; any stresses which di develop in it tended to cause big well-defined cracks, whereas the rubb would distribute and relieve its internal stresses more uniformly, whilst, necessary, local settlement could take place more easily, without suc serious consequences. Doubtless efficient piling and grouting had reduce the danger of unequal settlement to the minimum, but cavitation w always a possible danger in the course of years.

In design, the thickness of floor upstream could normally be muc less than that downstream from the aspect of uplift, since the uplift und the upstream floor was nil under working conditions, and its thickness wa therefore, controlled only by constructional considerations such as that unwatering. It was, therefore, much cheaper to extend the floor upstrea rather than downstream to obtain the necessary length required to preve dangerous seepage; that advantage was, however, in the past alwa offset by the advisability of having plenty of protection against sco downstream. With the increasing faith in the efficacy of the "kick-up wall at the extremity of the floor against that danger, the tendency future design should be to reduce the length of the downstream apro

That would give corresponding reduction in floor-thickness, and would reduce the depth of what often proved to be very troublesome excavation.

The piling, to be fully effective in developing its efficiency against "creep" or percolation, when placed in parallel rows, should be apart at least a distance equal, say, to twice the total depth of the rows of piling below the foundation. Closer spacing tended to encourage the diminution of the seepage length by "shorting" across the extremities of the piling; for economy, therefore, if the piles were placed closer than desirable, one or other of the set should be reduced in length. The Punjab Irrigation Research Institute had carried out some interesting experiments on that subject, and had published the results some years ago *.

Mr. Rotinoff had skilfully solved the problem of forming a continuous and effective concrete sheet-piling, and of avoiding the alternative, the difficulty of forming a deep concrete cut-off to the apron; he was fortunate, however, in having had a lengthy period in which to develop and acquire an invaluable technique of how to carry out the driving most efficiently. The contribution of that piling to the life of the structure was of immense value, although it was rash to state what its life would be compared with that of steel piling, as there were so many intangible factors.

What were assumed to be the worst conditions of stability for the eastern lock wall to enable the extension of width to be opportunely confined to the limit of the old sheet-piling? The connecting bars were presumably bent to facilitate ease of drilling and grouting the halves placed in the existing structure, and then of building the remainder into the new masonry, although clearly they were ideally best horizontal.

With regard to the paint-work for the steel, it was extraordinary that the protective qualities of tar were not more fully appreciated by engineers; the Admiralty were not afraid to use it, and Mr. Wilmot wondered if a complex existed against it owing to its relative cheapness! Its limitations were due only to its tacky nature.

**The Author,** in further reply to the Discussion and in reply to the Correspondence, pointed out that the length of the impermeable floor was determined on a head of 4·20 metres at Sharaki period and a hydraulic gradient of 1 in 20; no account was taken of the old cast-iron piles, as their efficiency as a cut-off was doubtful. The depth of the new reinforced-concrete piling was made similar to the depth of the cut-off at the Nag-Hamadi barrage. The heights of the baffle and deflecting weirs were determined from model-experiments carried out at the Delta Barrage Hydraulic Research Station. The creep theory for works of the description under discussion was adopted in Egypt, where it had proved satisfactory, and had not resulted in any failures.

---

* E. M. Taylor and H. L. Uppal, "A Study of the Flow of Water under Works on Sand Foundations by Means of Models." Punjab Irrigation Research Institute, Research Publications, vol. II. (1934), Nos. 3 and 4.

The remarks of Mr. Freeman were of interest as coming from a senior member of the contractors' staff.

The Author had referred in the discussion to the wide differences found in the hydraulic gradients inside the sudds. He considered that the chief value of the experiments was that they demonstrated the variations that had to be allowed for if safety were to be ensured. Both Mr. Norrie and Mr. Freeman mentioned that more water had had to be pumped from the downstream side of the sudds than from the upstream side at the end of each season. The Author considered that that was due to the large quantity of water which was seen to rise through that portion of the flexible apron on the downstream side which had been constructed during the previous season, and which was exposed to view inside the sudded area. That area was not blanketed with sand, as was the case on the upstream side, which was heavily loaded with the sand of the sudd, thus producing a stanching effect. Further, the area of the newly constructed flexible apron, through which the spring water rose freely on the downstream side, was nearly twice that of the upstream apron.

The Author considered that a well-constructed hand-packed rubble foundation under the flexible blockwork apron was preferable to gravel. That rubble at Assiut, in both the upstream and downstream aprons, was confined between the toes and the impermeable floors, so that the probabilities of its becoming disturbed or rounded by the erosive action of water were very remote indeed.

In practice it had been found that, if the rubble bed were packed in a proper manner, there was no settlement, and there was no difficulty in preparing a level bed on which to set the blocks. Moreover, at Assiut it was possible to make use of the old rubble, which was a considerable saving in cost to the contractors. Gravel, on the other hand, would be very liable to disturbance by traffic during construction, and the bed for each block as it was set would have to be screeded, thus increasing labour-costs; there would also be the risk of the smaller particles of gravel being washed out by spring water during construction, which would induce settlement.

The Author regretted that present circumstances would probably render it impossible to obtain information from Egypt to enable him to reply to some of the points raised during the Discussion.

He was unable to agree to some of the comments of Mr. Wilmot, and would refer him to p. 318 § for the relative amounts of cement used in grout below the upstream and downstream floors.

§ *Ibid.*

Paper No. 5230.

# "Highway Transition-Curves: A New Basis for Design." †

By Hugh Alan Warren, M.Sc. (Eng.), Assoc. M. Inst. C.E.

## Correspondence.

**Mr. R. K. Lewis** observed that, from a practical point of view, the engineer required an effective, clear, and simple formula which would enable a transition-curve to be designed or checked in the field, without reference to a book of tables.

Professor Royal-Dawson's * designs were based on what he called a "unit chord" which subtended an angle of 16 minutes at the pole; that standard made it difficult to follow and impossible to check his calculations without Tables.

The main point of difference between the two theories was the value that should be applied to $C$, the rate of change of radial (centrifugal) acceleration. Professor Royal-Dawson maintained that $C$ was of the utmost importance, and should not exceed 1 foot per second per second in a second. Mr. Warren, on the other hand, maintained that the value applied to $C$ was of minor importance and might be of any value between 0 and 15 feet per second per second per second.

Mr. Warren, in his experiments ‡, had proved beyond doubt that $C$ varied between wide limits, and that it was of greater importance to keep the value of $\frac{v^2}{Rg}$ (the amount of banking) to a minimum.

Professor Royal-Dawson, on the other hand, had no practical evidence for assuming that $C=1$. He had borrowed the value from Mr. W. H. Shortt, M. Inst. C.E. ‖ and applied it to roads, but Mr. Shortt had intended that value for railways; there was a considerable difference between the two. In practice, $C$ was evaluated by recording the track of a car running at a definite velocity and establishing the length of the major axis $a$ of the lemniscate. Then $C=\frac{3v^3}{a^2}$. It followed that the velocity would have to be recorded, but in the Toronto experiments referred to by Professor Royal-Dawson*, the velocities had not been recorded and, so far as $C$

† Journal Inst. C.E., vol. 14 (1939–40), p. 373 (June 1940).

* "Road Curves." Spon, London, 1936.

‡ H. A. Warren and E. R. Hazeldine, "Experimental Transition Curves." Journal Inst. Mun. & Cy. E., vol. lxv (1938–39), p. 1021 (March 28, 1939).

‖ "A Practical Method for the Improvement of Existing Railway-Curves." Minutes of Proceedings Inst. C.E., vol. clxxvi (1908–9, Part II), p. 97.

was concerned, the results were valueless. Again, the experiments under Professor Moyer, which were referred to by Professor Royal-Dawson ‡, had been carried out for the purpose of finding the maximum superelevation for skidding on circular arcs, and had no application to transition-curves.

**Mr. P. L. Pratley** congratulated the Author upon the form of the curve finally recommended. He agreed essentially with the statement of the Author that the basis of design should tend to be some limiting value for the vertical acceleration of the vehicle, and was fully sympathetic with the Author's feeling that the rate of increase of radial acceleration had very little practical effect. The value given by the Author to $C$, which was assumed to vary with the square of the velocity, while it actually expressed the third differential of space with respect to time, did not seem particularly logical, and some study had been given to the problem, using various examples, the result of which was an increased conviction that that feature could readily be eliminated as a criterion. The nature of the Author's curve and its derivatives was very intriguing, and the application of that curve should result in a very comfortable and satisfying transition. There were, however, one or two features to which Mr. Pratley would like to refer.

The expression for the vertical acceleration $\frac{d^2H}{dt^2}$, after having made the perfectly legitimate assumptions of $s = x$, and $\frac{1}{\rho} = \frac{d^2y}{dx^2}$, was written

$$\frac{d^2H}{dt^2} = \frac{m.w.v^4.k(L - 2x)}{g} \qquad . \quad . \quad . \quad . \quad . \quad (4a)$$

Two integrations of that equation yielded the curve for superelevation at the outside edge of the roadway, namely, $H = \frac{m.w.v^2.k.x^2(3L - 2x)}{6g}$, and the final value, at $x = L$, was $\frac{m.w.v^2}{g.R}$. Using equation (4) (p. 376 §) for the value of $k$ in the two examples given by the Author, that maximum superelevation, or the superelevation of the circular curve, became 2·90 feet and 2·68 feet, assuming in both cases a 30-foot roadway and the quoted value for $m$ (namely, 0·4). A third example would be quoted which corresponded with the local practice in Montreal, where $v$ was taken as 40 miles per hour, $R$ as 500 feet, $w$ (the width of the roadway) as 24 feet, $p$ as one-quarter of gravity (or 8 feet per second per second), and $m$ as $\frac{1}{2}$. The superelevation for that third case worked out at 2·06 feet for the circular arc. The length of transition curve was 81·8 feet, and the time

‡ F. G. Royal-Dawson, "Common Fallacies regarding Road Curvature." Journal Inst. Mun. & Cy. E., vol. lxv (1938–39), p. 837 (January 31, 1939).

§ Page numbers so marked refer to the Paper. Footnote (†), p. 579.—Sec. Inst. C.E.

of transition was about 1·39 second, using only the first criterion, namely, that stated in equation (5) (p. 378 §).

The final angular acceleration reached during the transition was that appropriate to the circular curve, namely, $\frac{v^2}{R}$, and the average rate of acquiring that acceleration was $\frac{v^3}{L.R}$; it was not acquired uniformly, however, as the Author pointed out, and the maximum rate of acquiring the acceleration was 50 per cent. more than the mean rate, or $\frac{3v^3}{2L.R}$. That corresponded precisely with equation (6) (p. 378 §), and the application of that equation to the three examples wherein the length $L$ had already been established by the criterion, showed that the value of $C$ was reasonably constant and very close to 8 feet per second per second per second.

It would seem reasonable that if $C$ were to be adopted as a criterion, a maximum value of about that figure should be set up instead of a value depending upon the square of the velocity; but the introduction of a second criterion based upon that conception of a value for $C$ served no useful purpose, as the results in each example would be to reproduce a value for $L$ approximately the same as that obtained from the use of the first criterion, which was the condition regarding vertical acceleration whereby $p$ was limited to one-quarter of gravity. Therefore, although it would be convenient to establish $C$ so as not to exceed 8 feet per second per second per second, the procedure would seem almost useless. It was, however, of interest to note that with $C = 8$, $L = \frac{3v^3}{16R}$, and that was, therefore, the minimum length advisable from considerations of the second criterion. To make the two criteria equal, meant establishing a relation between speed and curvature which, if $m$ were retained as equal to 0·4, could be expressed as $v^2 = \frac{w.R}{8}$, when $v$ was the speed in miles per hour. That relation was again almost useless, as none of those factors $v$, $w$, or $R$, could be chosen merely from those considerations, but it might be useful as a general relationship to be borne in mind when the choice of the radius of the circular curve was being made.

Mr. Pratley thought, therefore, that the criterion expressed in equation (5) (p. 378 §) was by itself ample for the purpose advocated, and that rates of turning the steering wheel were never sufficiently serious to become predominant or even to be introduced into the problem. The curve was easy to set out both horizontally and vertically, and its adoption should be a very real contribution to smooth riding and easy manipulation.

It would appear that in setting-out by deflexion-angles, the ratio

§ *Ibid.*

$\tan\theta/\tan\phi$, at $x = 0$ was 1/4, and not 1/5 as given on p. 379 §, the expression being $\frac{(5L - 2x)}{10(2L - x)}$.

It was rather unfortunate, and somewhat confusing, that $m$ and $w$ should have been chosen to represent an arbitrary ratio and the width of roadway respectively, as in most systems of nomenclature they had other definite significations.

**Mr. L. R. Robertson** observed that Mr. Warren's mathematical experiment in the evolution of an entirely new transition-curve was very interesting, but it was doubtful whether it would be of great practical value. Mr. Warren seemed to attach considerable importance to the rigid linking-up of the shape of the transition-curve with the shape of the longitudinal profile showing the rate of application of the superelevation; as, however, the actual superelevation was only an arbitrary fraction (usually 0·4) of the theoretically correct superelevation, it was difficult to see the necessity for such a rigid linking-up. There was no practical difficulty in the rounding-off of the superelevation at the ends and centre of the curve—it was a matter of a very few minutes with some French curves in the drawing office, and the most important consideration in that connexion was surely to obtain a pleasing curb-line. It was Mr. Robertson's experience that in an open sweep of road it was impossible to obtain that if the whole superelevation was applied in a distance of 130 feet, as in Mr. Warren's example 1 (p. 381 §). Unless the change from the cambered or crossfall section to the final superelevated section of the road occupied several hundred feet, the curb-line might appear distinctly displeasing.

Mr. Warren based a large part of his calculations on the vertical acceleration of a car travelling along the tilting road, but it was more usual to tilt a road about its centre-line than about one of its curbs, and, moreover, a car at speed was usually nearer to the centre-line than to the curb, so that the vertical acceleration was small, and did not enter into the transition problem.

The most controversial point in the Paper, however, was probably the value of $C$ that he proposed, which governed the length of the transition-curve. Mr. Shortt's experiments ‡ established that $C = 1$ gave a comfortable rate of transition on railways; on roads, owing to the greater bumpiness of the motion in general, it was probable that a slightly higher value of $C$ would pass unnoticed, perhaps up to $C = 2$; but could it really be assumed in Mr. Warren's example 1, if the limitation due to vertical acceleration were negligible owing to the road being a fairly narrow one and tilted about its centre-line, that he would seriously suggest a transition-length of 13·2 feet, with a value of $C$ of over 50? That would involve applying a centrifugal force of 35 lb. to a 10-stone

§ *Ibid.*

‡ "A Practical Method for the Improvement of Existing Railway-Curves" Minutes of Proceedings Inst. C.E., vol. clxxvi (1908–9, Part II), p. 97.

passenger in about $\frac{1}{7}$ second! The value of $C$ to be used in design depended on several factors, such as the comfort of the passengers, suitable rate of turning the steering-wheel, the risk of the vehicle skidding, etc. With regard to the first factor alone, Mr. Robertson could not accept as ideal, in the design of a road which should still be satisfactory in 40 years' time as well as at present, any value of $C$ greater than 2, and there was no reason, where space was available, why the really comfortable standard of $C = 1$ should not be used. When designing a road, it was not so much the fresh and alert driver, but the driver or passenger who had already been on the road for several hours, who should be considered; and apart from the mental concentration required by the driver for high values of $C$, the sideways rolling motion produced by the constant application of $C$ first from one side and then from the other tended to produce car-sickness in the passenger. When $C = 2$, a sideways force of a quarter of the weight of the passenger, less a fraction according to the amount of superelevation provided, was developed on him in 4 seconds (or in 8 seconds for $C = 1$) which probably gave him time to adjust himself to the rolling tendency without a conscious effort each time; but that would not be the case with higher values of $C$.

Finally, Mr. Robertson wished to point out that the maximum difference between Mr. Warren's transition-curve and the lemniscate, each 130 feet long, and each of 1,000-foot final radius, as in his example 1, was just over 3 inches. In a carriageway of 25- or 30-foot width, a difference of 3 inches was negligible, provided that a pleasing curve was obtained, so that once the all-important length of a transition-curve had been fixed, it appeared to be a secondary matter which particular mathematical transition-curve was used. The curve should be chosen largely according to the ease with which it could be calculated and set out, and the lemniscate was still preferable in that respect; for the deviation angle of the curve at any point was always exactly three times the polar deflexion from the tangent-point, and very full tables of lengths and angles had been compiled for the practical use of the lemniscate.

**Professor F. G. Royal-Dawson** observed that the Author's proposals, although supported by abstruse mathematics, were unfortunately based on fallacious premises. The Author had relied on the supposed results of certain experiments. Professor Royal-Dawson had analysed those experiments, and a discussion arose * therefrom. In the light of the present Paper, the main points at issue might be reviewed in their true perspective.

Briefly, the Author's initial fallacy lay in assuming that the advocacy of the $C = 1$ standard, as a basis for road-curve design, implied that

* H. A. Warren and E. R. Hazeldine, "Experimental Transition Curves." Proceedings Inst. Mun. & Cy. E., vol. lxv (1938–39), p. 1021 (March 28, 1939). Correspondence on this Paper, p. 1104 (April 11, 1939), p. 1146 (April 25, 1939), p. 1192 (May 2, 1939).

$C = 1$ represented the "maximum comfortable" rate of turning (that was to say, just short of discomfort). In order to test that assumption, he performed experiments with light cars at low speeds in a car park which, being a *cul-de-sac*, could not possibly have represented road conditions. Consequently, as the experimenters had no objective, they simply made blind turns in any direction at haphazard speeds ranging from 10 to 37·5 miles per hour; in most cases a maximum centrifugal ratio of 0·25 being attained in approximately a second. The sole idea was apparently to test their individual conceptions of "comfortable" or "natural" turnings in a light car. As might have been expected, all sorts of values of $C$ were obtained, ranging, by the Author's computation, from 0·7 to 34·1 and even 58·4, the last two figures being obvious miscalculations, since Professor Royal-Dawson's analysis disclosed no figure higher than 11·6. The Author's final verdict † on those experiments was that "having travelled in perfect comfort over transitions of characteristic $C = 10$ and the like, it was not easy to continue reading statements that $C = 1$ is the maximum comfortable value, without some protest." He even presented ‡ the curious corollary that "present designs based on $C = 1$ are just so much scrap." The logic of that remark was not evident, since, if it were deemed "comfortable" to attain a centrifugal ratio of 0·25 in 1 second, it would be even more comfortable to attain it in 8 seconds (the $C = 1$ standard). That was, therefore, no argument for "scrapping" the latter, but rather the reverse.

The Author did not mention where he had read the statements to which he referred. They did not occur in Professor Royal-Dawson's writings. On the other hand, the latter's advocacy of the $C = 1$ standard as the basis for road-curve design did not rest on any individual's idea of the maximum limit of comfort, but on a much wider foundation.

On the open road the motorist had a definite objective. He was concerned only with reaching his destination at his normal rate of speed with the least amount of exertion, never hesitating to take the line of least resistance (for example, cutting corners) as opportunity offered, and with no thought of trying centrifugal experiments. That was a matter of common observation, and the Toronto records * clearly revealed to any unprejudiced mind that the routine and unpremeditated movements of motorists round a curve complied unwittingly with a standard approximating to $C = 1$. Moreover, road traffic included not only light cars, but also heavy lorries, motor-coaches, and ambulances, at all speeds, in all of which dynamic considerations affecting the safety of the vehicle and its contents in transit had to be taken into account. Even without the

† *Loc. cit.*, p. 1146.
‡ *Loc. cit.*, p. 1026.
* F. G. Royal-Dawson, "Speed in Relation to Curvature, with Special Reference to Road-Curves." Minutes of Proceedings Inst. C.E., vol. 240 (1934–35, Part 2), p. 132.
F. G. Royal-Dawson, "Road-Curves." Spon, London. 1936, p. 172.

direct evidence of the Toronto records, it would be evident to anyone having a clear conception of the meaning of $C$ that, the lower the value of $C$, the less was the fatigue imposed on the driver, who consequently had less temptation to cut corners. Thus the $C = 1$ basis stood on its own merits as the safest and easiest that could be devised.

Reverting to the present Paper, the Author's claim to have proved from those experiments that $C = 1$ was very far from practical reality as a basis of road-curve design, was completely controverted by the facts of the case as set forth above. Such experiments had not even touched the fringe of the subject, and might from that point of view be regarded as "just so much scrap," for their avowed object was to find the highest values of $C$, within their range of speeds, short of discomfort. The utmost that they could prove, if proof were necessary, was that light cars at low speeds had a certain amount of swerving power in emergencies.

More serious, however, than the mere futility of those experiments was the Author's misinterpretation of them, due to a fallacious conception of the meaning of $C$. His main arguments had been invalidated throughout by using that symbol in a wrong sense.

The symbol $C$ as used by Mr. Shortt * had a definite and precise meaning. It might be defined as the number of feet (or other linear units) per second per second in a second, denoting rate of gain of centripetal acceleration. Its basic equation was $C = \frac{v^3}{RL}$, where $RL$ was a constant transition. Thus, for a given transition, $C$ varied as $v^3$. Conversely, $C$ had no meaning without reference to a given transition.

The Author's first violation of that definition occurred † when he attempted to relate the growth of $\frac{v^2}{r}$ with time, to the rate of turning the steering wheel (the front wheels being turned to an angle $\alpha$, whilst $B$ was the wheel-base of the vehicle). He said, "the rate of increase $\frac{v^2}{r} = \frac{d}{dt}\left(\frac{v^2}{r}\right)$ which will be denoted by $C$." Whence, substituting $r = \frac{B}{\alpha}$, he obtained

$$C = \frac{d}{dt}\left(\frac{v^2\alpha}{B}\right) = \frac{v^2}{B} \cdot \frac{d\alpha}{dt}.$$

It did not require much calculation to perceive that when a vehicle proceeded at a steady speed while the steering wheel was turned at a steady rate, a true transition-curve was automatically described, but as, with a given rate of turning the steering wheel through an angle $\alpha$, the scale

---

* A Practical Method for the Improvement of Existing Railway-Curves." Minutes of Proceedings Inst. C.E., vol. clxxvi (1908–9, Part II), p. 97.

† H. A. Warren, "Highway Transition Curves." Proceedings Inst. Mun. & Cy. E., vol. lxv (1938–39), p. 302 (August 5, 1939).

and extent of the transition described depended entirely on the vehicle-speed, varying as the latter varied, it was impossible to relate such diverse factors in general terms to the symbol $C$, which by its very definition was determined solely by the factor of vehicle-speed in relation to a given transition.

Unfortunately that misuse of the symbol $C$ was carried a step farther †, and arbitrary values were assigned to $B$ and $\frac{d\alpha}{dt}$. Taking $B$ as 7·86 feet and $\frac{d\alpha}{dt}$ as 4·5 degrees per second (or about 27 degrees per second on the steering-column with a gear-ratio of about 6), the formula became $C = \frac{v^2}{100}$.

Thus, by specious reasoning, invoking arbitrary steering details, a spurious meaning had been attached to $C$, making it appear as a function of $v$ only, varying as $v^2$ without reference to any other consideration, whereas the real $C$ varied as $v^3$ with reference to a given transition.

TABLE I.

| Speed: miles per hour. | 10 | 15 | 20 | 25 | 30 | 37·5 |
|---|---|---|---|---|---|---|
| $C$ (by Author's formula) . | 2·1 | 4·9 | 8·6 | 13·4 | 19·3 | 29·4 |
| $C$ (by experiment, Author's figures) . | 0·79 | 3·91 | 13·6<br>18·0<br>4·34 | 34·1<br>5·24 | 13·9<br>5·16 | 58·4 |
| $C$ (by experiment, plotter test) . . . | 1·0<br>4·7 | 3·0 | 8·2<br>7·0<br>5·0 | 9·9<br>7·0 | 7·2<br>5·4 | 11·6 |

The erratic nature of the values of $C$ assigned by the Author to the various experiments were due to his crude method of extracting them, the chief fallacy being the assumption that the transition began exactly on the zero cross-line of observation (the approach-lines not being recorded). Professor Royal-Dawson's method of analysis was to superimpose a transition-plotter of a size selected in each case to fit accurately with the curve from the observation zero to the crucial part of the curve approaching the minimum radius. That revealed the true starting point of the transition, occurring in most cases on the approach prior to reaching to observation zero mark. Table I gave the comparative figures (in which grouped figures had approximately the speeds named in their respective columns). Whether the Author's or Professor Royal-Dawson's figures on the experi-

† H. A. Warren and E. R. Hazeldine, "Experimental Transition Curves." Journal Inst. Mun. & Cy. E., vol. lxv (1938–39), p. 1025 (March 28, 1939).

ments were accepted, it required more than a stretch of imagination to see any correspondence between them and the Author's formula, which was not surprising, as there was no logical relationship between them, since at least half a dozen different sizes of transition were involved indiscriminately in ten experiments, and for the same speed two or more widely varying figures were obtained in three cases.

Incidentally, the experiment at 37·5 miles per hour, in which a record centrifugal ratio of 0·33 was achieved in less than a second, was described as a "natural turn," whatever that might mean.

Arising from that spurious conception of $C$, the Author's final inference was that "if the speeds had been any higher so would the value of $C$ obtained, and the transition part of the curve would have shrunk away to nothing ‡."

TABLE II.

| Speed. | | Transition, $RL$. | For centrifugal ratio 0·25. | | | $C$: feet per second per second in a second. |
|---|---|---|---|---|---|---|
| $v$: feet per second. | $v$: miles per hour. | | $R$: feet. | $L$: feet. | Time of transit: seconds. | |
| 14·67 | (10 ) | 1,467 | 26·6 | 55 | 3·75 | 2·1 |
| 22·0 | (15 ) | 2,200 | 60·0 | 36·66 | 1·66 | 4·9 |
| 29·34 | (20 ) | 2,934 | 106·6 | 27·5 | 0·94 | 8·6 |
| 36·66 | (25 ) | 3,666 | 166·6 | 22·0 | 0·510 | 13·4 |
| 44·0 | (30 ) | 4,400 | 240·0 | 18·33 | 0·417 | 19·3 |
| 55·0 | *(37·5) | 5,500 | 373·6 | 14·72 | 0·267 | 29·4 |
| 58·66 | (40 ) | 5,866 | 426·6 | 13·75 | 0·234 | 34·5 |
| 73·33 | (50 ) | 7,333 | 666·6 | 11·0 | 0·15 | 53·5 |
| 88·0 | (60 ) | 8,800 | 960·0 | 9·16 | 0·104 | 77·1 |

* Maximum speed reached in the Author's experiments.

The absurdity of that reasoning could best be illustrated by equating the spurious formula to the true one. Thus $C = \frac{v^2}{100} = \frac{v^3}{RL}$, whence $v = \frac{RL}{100}$.

The results of that equation were given in Table II.

Thus whilst, at a speed of 10 miles per hour, the time allowed for attaining a centrifugal ratio of 0·25 was 3·75 seconds, a vehicle (perhaps a heavy motor-coach) travelling at 60 miles per hour was to be jerked into the same centrifugal ratio in $\frac{1}{10}$ second! The lengths of transition were 55 and 9 feet respectively, and the corresponding values of $C$ were shown in the last column. Those were the values supposed to have been confirmed by the Author's experiments.

‡ H. A. Warren and E. R. Hazeldine, "Experimental Transition Curves." Proceedings Inst. Mun. & Cy. E., vol. lxv (1938–39), p. 1146 (April 25, 1939).

By way of contrast, if the $C = 1$ standard were consistently ar correctly used, the above-mentioned coach, travelling at 60 miles pe hour over the same minimum radius of 960 feet, would be allowed 8 second to travel over a transition 720·4 feet long. That was what the Autho called very far from practical reality.

In the face of such anomalies, it was no wonder that the Autho expressed misgivings as to whether the conception of $C$ was of itself suitable basis of design at all. His conception of it was certainly no suitable basis. His present proposal to consider superelevation as a ne basis arose from his dilemma in having to provide, as the result of hi practical extermination of the transition-curve, an expedient for th almost instantaneous application of a full centrifugal ratio of, say, 0·25 He recognized that superelevation applied too abruptly (even by th 0·4 rule) involved a steep short longitudinal gradient, which would hav to be eased by vertical curves at the changes of gradient. To compensat for those vertical curves, the transition-curve was partially reinstated but was no longer a true transition, but one which, by meticulous calcula- tion, required to be flatter than it should be at the ends. In spite of that having in view the shortness of the transition, and the consequent abrupt- ness of the superelevation, he visualized a sudden application of vertica acceleration, the effect of which was to be " absorbed by tire-cushioning springing, etc." The Author recognized that for high speeds on a shor transition the vertical acceleration was the main consideration. Beyond specifying, however, that it should not exceed $p$ feet per second per second he gave no hint as to how to determine the maximum permissible value of $p$ in a given case.

The Author gave two alternative methods of determining the length of transition : (1) by the vertical acceleration formula, using an arbitrary value for $p$ ; (2) by the so-called " $C$ " formula, applied to his specia form of transition.

His numerical example 1 might now be analysed. It was required to join a straight to a radius of 1,000 feet, with a transition fit for 60 miles per hour, given a road-width of 30 feet, and that $p$ was not to exceed 8 feet per second per second.

By method (1) he obtained $L = 130$ feet, traversed in 1·5 second.

By method (2) he obtained $L = 13{\cdot}2$ feet, traversed in 0·15 second.

Then, by way of comparison, he tried the effect of using $C = 1$, by which he erroneously obtained $L = 1{,}024$ feet, traversed in 11·65 seconds (That could be shown to correspond to $C = 0{\cdot}666$).

The true formula, using $C = \frac{v^3}{RL}$ and making $C = 1$, gave $L = 682$ feet traversed in 7·75 seconds.

The corresponding values of $C$ by the true formula would be (1) $C = 5{\cdot}25$ ; (2) $C = 53$ ; (3) $C = 0{\cdot}666$ ; (4) $C = 1$.

The following further analysis might be noted : the maximum centri

ugal ratio was 0·24; hence the banking at edge of 30 feet was, say, 3 feet. Thus the longitudinal gradient in the four cases was: (1) 1 in 43·3; (2) 1 in 4·4; (3) 1 in 341·3; (4) 1 in 227·3; whilst the vertical acceleration $p$ was (1) 8; (2) 774; (3) 0·2; (4) 0·29 feet per second per second. Hence, in gradient as well as in vertical acceleration, the advantages of the fourth method over the first method were very great. In the fourth method the gradient was so easy as to preclude the necessity for considering vertical acceleration, or the Author's special form of transition-curve, because the usual vertical curves given by highway engineers as a matter of course at all changes of gradient removed the necessity for special shock-absorption devices on the car.

In conclusion, reviewing the whole question, it would seem that the Author, in endeavouring to express $C$ in terms of an arbitrary steering factor, had assumed that a steering-wheel could be turned to any given angle, at any rotary speed, "lightning" or otherwise, at any given moment at will, without regard to physical possibilities or to the reaction on the vehicle and its contents.

The existence of such factors as momentum of the vehicle concerned, back-lash in the steering mechanism, and castor action tending to counteract the steering effort, also the effect of a sudden lateral tilt on the springs and contents of a heavy vehicle due to an over-brusque rate of super-elevation, not to mention skidding and overturning tendencies, did not seem to have occurred to the Author, for otherwise he might have realized that his formula was fundamentally unworkable.

On the question of momentum alone, it was only necessary to compare the kinetic energy, $\frac{Wv^2}{2g}$, of a 1-ton car at 20 miles per hour, with that of a 10-ton vehicle at 60 miles per hour, to note that the latter was ninety times as great as the former, with a speed only three times as great. To that should be added the kinetic energy of the rotating wheels, which might amount to 10 per cent. more, tending to carry the vehicle forward tangentially at any moment against any steering effort.

Moreover, without assigning numerical values to the steering effort required for a given vehicle in example 1 (60 miles per hour on a 1,000-foot radius), it would be obvious that in comparing the first and fourth solutions, where the turns were made in 1·5 second and 7·75 seconds respectively, the steering effort (by the principle of work and energy) would have to be more than five times greater in (1) than in (4). The required effort in solution (2) might be left to the imagination. Put in another way, the wrist-force required to turn the steering-wheel through a given angle in 7·75 seconds would have to be increased fivefold at least to make the turn in 1·5 second. That alone was a sufficient argument for the longer transition. It also showed the futility of conducting experiments with the avowed object of ascertaining the "maximum comfortable" rate of turning, for whilst a swerve to a centrifugal ratio of 0·25 in less than a

second might be the Author's ideal of "perfect comfort", a road-rac
skidding round a hairpin bend might also consider himself "comfortable
so long as his car did not actually overturn.

In short, the more deeply the question of road transitions was studie
the more convincing did it become that the only rational basis of roa
curve design was a low value (the lower the better) of the symbol
correctly applied to the speed standard required, as that ensured th
minimum of fatigue to any driver at any speed, especially at high speed
Professor Royal-Dawson * had always allowed for a permissible excess
speed over the $C = 1$ standard for light cars at low speeds, according to
sliding scale.

**The Author**, in reply to the Correspondence, stated that a great deal
rather tangled theory and philosophy had grown up on the subject of
suitable, natural, or comfortable value of $C$, all of which might have bee
avoided by conducting a few simple and direct experiments with actu
road vehicles. Such experimental work the Author had carried out, bu
Professor Royal-Dawson, in his communication, sought to dispute th
results, and weaken the conclusions thus obtained. The Author woul
merely say that until Professor Royal-Dawson could put forward direc
experimental evidence of his own, obtained from recorded tracks of roa
vehicles, there existed no basis of discussion. It was such a simpl
matter for any surveyor to carry out the experiments described † and t
check the conclusions obtained, that any further philosophical discussio
on that point was a waste of time. The secondhand evidence on whic
Professor Royal-Dawson relied was completely worthless for reasons se
out in the communication from Mr. S. K. Lewis. One could not but fe
that had Mr. Shortt ‡ originally recommended a value for $C$ of 10 feet pe
second per second per second, Professor Royal-Dawson would with equ
facility have obtained that figure and supported that value.

The intrinsic meaning of $C$ was, of course, the rate of gain of centrifug
acceleration, but the Author was of the opinion that, so far as comfort an
safety were concerned, limitations to its numerical value on those groun
were so wide that other secondary considerations became predominan
for instance, the rate of turning the steering-wheel, and the vertic
acceleration arising from banking the surface. Consideration of the ra
of turning the steering-wheel gave $C = \frac{v^2}{100}$ on the assumption that t
wheel was turned at 27 degrees per second, although Professor Roya
Dawson saw in that a "violation of the definition of $C$," presumab

* "Elements of Curve Design." Spon, London, 1932, pp. 104, 138. "Ro
Curves." Spon, London, 1936, pp. 172, 242.

† H. A. Warren and E. R. Hazeldine, "Experimental Transition Curve
Journal Inst. Mun. & Cy. E., vol. lxv (1938–1939), p. 1021 (March 28, 1939).

‡ "A Practical Method for the Improvement of Existing Railway-Curve
Minutes of Proceedings Inst. C.E., vol. clxxvi (1908–9, Part II), p. 97.

because he was unable to conceive other than a fixed value for $C$. His statement that for a "given transition" the value of $C$ experienced would be proportional to $v^3$ was certainly correct, but, like so much of his communication, was irrelevant, since no "given transition" was concerned. The Author was not perturbed that his experimental values were erratic and differed from $\frac{v^2}{100}$, since that was just the conclusion he had arrived at, namely, that the value of $C$ could and did vary within the widest limits. Professor Royal-Dawson was again only demonstrating the need for a basis other than $C$ when, subsequently to his Table II, he used the values he had obtained for the time of transit in an endeavour to pour ridicule on the use of $\frac{v^2}{100}$ as a value for $C$. In both the numerical examples set out by the Author, the limitation to length, and therefore time, of transit, arose not from the $C$-basis at all, but by reason of vertical accelerations. That was the chief reason for the creation of a new type of curve.

Professor Royal-Dawson had pretended to find mistakes in the Author's first example by the artifice of calculating $L$ from $L = \frac{v^2}{R.C.}$, which he referred to as the "true formula." He had stated that he found the mathematics "abstruse," and was therefore possibly unaware that his "true formula" was correct only for the spiral, and that for the lemniscate, the cubic parabola, and the suggested curve it was far from being true on account of the rate of gain of curvature not being constant. His further remarks concerning momentum and kinetic energy were redundant since those were only alternative conceptions of the effect usually expressed as centrifugal force. The next point raised about the wrist-force required to turn the steering-wheel would be seen in true proportion if it were noted that, for a 1000-foot radius, assuming a wheel-base of 7·6 feet, and a steering ratio of 6 to 1, the angle turned through by the column was 2·61 degrees. Speculations and philosophy on the wrist-force required to achieve that momentous task were thus somewhat idle. At high speeds greater difficulty was experienced in keeping the wheel straight than in turning it through such small angles. In direct contradiction to Professor Royal-Dawson's last remark, the reasonable rate of turning the steering-wheel did become a limiting factor at very low speeds when the angle that could be turned through before $\frac{v^2}{R}$ exceeded 8 feet per second per second was large, but such small radii were outside the scope of the highway engineer.

The Author had found it very helpful and encouraging to read the communication from Mr. Pratley. The expression of the allowable value of $C$ in terms of $\frac{v^2}{100}$ followed directly from the assumption of a reasonable figure for the maximum rate of turning the steering-wheel, and should be

taken as only a rough guide. Mr. Pratley had noted that in both the Author's examples, and in one cited by himself, the value of $C$ actually experienced on the transition designed was approximately constant at 8 feet per second per second per second. The Author had investigated that point, and if, as Mr. Pratley had suggested, the value of $L$ were found solely from considerations of vertical acceleration, that is, from $L = v^2 \sqrt{\frac{6m.w}{R.g.p}}$ and the resulting value of $C$ was found from $C = \frac{3v^3}{2R.L.}$, then, substituting for $L$, it followed that $C = \sqrt{\frac{3gp}{6mw} \cdot \left(\frac{v^2}{R}\right)}$. Using the values stated to be the local practice in Montreal, it was true that $C$ was found to be approximately 8 feet per second per second per second, which verified in general terms what Mr. Pratley had found by numerical examples. The Author was inclined to agree that for most practical highway-curves the conception of $C$ was no longer a basis for design; indeed, that was the main purport of his Paper, but, on the other hand, if $C$ were to be retained as a check criterion, the Author was not entirely prepared to accept a fixed value of 8 as Mr. Pratley had suggested, since although that figure was suitable for the average conditions stated to be the practice in Montreal, it might not be so suitable for the design of, say, a clover-leaf crossing with sharp radii and for low speeds; in such cases a lower value of $C$ would be more suitable, since at low speeds the maximum reasonable rate of turning the steering-wheel became the predominant criterion.

Mr. Pratley had rightly drawn attention to a numerical error in the Paper regarding the deflexion-angle, and the Author had to thank him for that correction.

Mr. L. R. Robertson, evidently after mature consideration, had raised some interesting points. The shape in plan of existing transition-curves had been governed by the consideration that the rate of gain of radial acceleration should be finite and more or less constant (exactly constant in the case of the spiral). If that consideration were no longer important, as the Author believed his experimental work to have proved, then the next predominant criterion of comfort and safety, namely, vertical acceleration, should logically govern the shape of the curve. The Author was willing to admit that the superelevation need not be linked to the curvature of the transition by a constant ratio such as 0·4, but if that method were adopted, then the transition would have the merit of being mechanically perfect for vehicles travelling at $\sqrt{0{\cdot}4}$ of the design speed. Regarding the æsthetic value of the curb-line, the Author would have thought that the smoothly curved longitudinal profile of the new curve would have provided, even in short lengths, just the pleasing effect that Mr. Robertson so desired, but he was not prepared to be dogmatic about it. On the point concerning the tilting of the road-surface, the Author, realizing that there

was diversity of practice in that respect, was careful to frame the formulas with the symbol $w$ denoting either the full width of the road when tilted about one channel or the semi-width when tilted about the centre-line, and the numerical calculations set out were equally valid for either type. Admittedly, vehicles travelling at speed were often nearer the crown than the curb, but there, as in all design, one had to allow for the worst case, and the maximum vertical acceleration would naturally be experienced when travelling around the outside of the banking. Mr. Robertson expressed misgivings at the use of high values of $C$, but if he were to carry out a few simple experiments on the lines previously mentioned the Author was confident that Mr. Robertson would become convinced that in so far as highways were concerned, the experience of Mr. Shortt * in 1906 regarding railway-curves was only interesting from a historical point of view. The example quoted, in which a force of 35 lb. operated on a 10-stone passenger in $\frac{1}{7}$ second, certainly sounded disturbing; stated in another way, however, the resultant force on the portion of the body above his seat became inclined at angle of 14 degrees to the vertical, and therefore still passed well within his "contact area", thus maintaining stability without any conscious effort on the part of the passenger; the effect then appeared less alarming. It should be remembered that the 35 lb. quoted was not a concentrated force, but was distributed all over the body, and only the portion above the seat tended to cause sway. In self-defence the Author had to point out that the $\frac{1}{7}$ second quoted above did not correspond to the solution used in his numerical example; but, for the hypothetical case stated by Mr. Robertson, the Author was quite convinced that $\frac{1}{7}$ second was not incompatible with comfort or safety, both the latter being dependent on the absolute value of $\frac{v^2}{R}$ and almost independent of the rate of gain of $\frac{v^2}{R}$. With regard to the point raised on tired drivers and the "mental concentration" necessary for high values of $C$, the angle to be turned by the steering-column was, as stated before for that example, only 2·61 degrees, and the Author could not believe that that required such a mental effort as had been suggested; indeed, more concentration would be required to occupy 11·65 seconds over that task, corresponding to the $C = 1$ standard. Regarding car-sickness, any continued rolling from one side to the other would be the result of reversal of curvature, namely $\frac{v^2}{R}$, and had little or no connexion with values of $C$, high or low.

Admittedly there was not a wide divergence in plan between the new curve and the lemniscate, but the Author was not prepared to admit that the latter was in any way easier to set out, or to calculate; and in any case it was surely a bad principle to select a certain curve merely because it made the calculations easier. The recommended curve corresponded to

* "A Practical Method for the Improvement of Existing Railway-Curves." Minutes of Proceedings Inst. C.E., vol. clxxvi (1908–9, Part II), p. 97.

more natural movements of the steering-wheel than did the lemniscate, and was therefore more likely to agree with the path of the vehicle.

It was most gratifying to read the kind remarks made by Mr. S. K. Lewis, and the Author hoped that other practical highway engineers would give the new curve a fair trial, even if their adherence to the old order induced them to use rather lower values of $p$ and $C$ than those suggested. It should be pointed out, however, to those who believed that by using lower values of $C$ they were designing safe roads, that, for a given total deviation-angle and fixed tangent-points, or mid-points, the lower the value of $C$ used the less would be the minimum radius of the bend, whether all-transitional or whether compounded with a circular arc. In other words, by restricting the rate of gain of radial acceleration, the final absolute value had to be greater; consequently real comfort and safety, as expressed in a minimum value of $\frac{v^2}{R}$, was in that way sacrificed for an illusory gain supposed to be associated with low values of $C$.

# LATE CORRESPONDENCE ON PAPER PUBLISHED IN NOVEMBER 1938 JOURNAL

Paper No. 5127.

## "The Protection of Dams, Weirs, and Sluices against Scour." †

By Robert Valentine Burns, Ph.D., B.Sc., Assoc. M. Inst. C.E.,
and
Assistant-Professor Cedric Masey White, Ph.D., B.Sc.

### Correspondence.

**Mr. A. R. B. Edgecombe** observed that Dr.-Ing. Theodor Rehbock, in his contribution to the Correspondence on the Paper*, after pointing out, quite correctly, that there were more weirs and dams in British India than in any other country, expressed surprise that it was one of the few countries where his dentated sill was not used. He attributed that fact to the action of the Central Board of Irrigation in distributing throughout India, by their official publications, Reports of tests carried out in the Punjab Irrigation Research Institute at Lahore which showed that the Rehbock dentated sill was unsatisfactory. Dr. Rehbock stated that the tests in question had not been performed correctly.

† Journal Inst. C.E., vol. 10 (1938–39), p. 23 (November 1938).
* Journal Inst. C.E., vol. 12 (1938–39), pp. 266 *et seq.* (October 1939).

In the first place, Mr. Edgecombe would point out that no official publication of the Central Board of Irrigation contained either a description or the results of the tests with Dr. Rehbock's dentated sills carried out at the Punjab Irrigation Research Institute, nor had any opinion been expressed in any of the publications against the use of dentated sills. Opinions expressed by certain Research Officers on their unsatisfactory experiences with dentated sills, in the course of discussions at the annual meetings of the Research Committee of the Central Board of Irrigation, had been published in the Annual Technical Reports of the Board, but they were clearly recorded as expressions of individual opinions, and could in no way be construed as the views of the Board. In point of fact, one of the Board's important technical publications ‡ listed Dr. Rehbock's dentated sill as one of the devices for dissipating energy downstream of weirs along with other devices serving the same purpose, and a description of the dentated sill in Dr. Rehbock's own words was quoted from Freeman's "Hydraulic Laboratory Practice,"‖ with illustrations showing the correct position of the sill.

A brief reference to tests of dentated sills carried out with unsatisfactory results at the Punjab Irrigation Research Institute, Lahore, in 1936 was contained in an abstract published in one of the Quarterly Bulletins issued by the Board. The Quarterly Bulletins were, however, mere collections of abstracts of all literature received in the Board's library, and did not in any way reflect the opinions of the Board.

With regard to Dr. Rehbock's observation that many Indian engineers were unaware of the criticism that he had made of the tests with dentated sills made in India, it might be of interest to Dr. Rehbock to know that his criticism of the allegedly incorrect experiments with dentated sills carried out by Dr. H. L. Uppal and Mr. N. N. Bhandari††, was published in abstract form in an issue of the Board's Quarterly Bulletin, which was widely distributed throughout India.

Dr. Rehbock contended that the expense incurred in lowering a layer of concrete blocks 60 feet broad by 4,000 feet long through a distance of 4 feet, on the Marala weir in the Punjab, could have been avoided by the installation of dentated sills, as his model-tests had clearly shown.

The reason why it was necessary to reconstruct the downstream glacis and floor of the Marala weir, and to lower the latter as well as the loose protection beyond by 4 feet, was that in high floods such as those of 1929 and 1930, the standing wave formed on the semi-pervious floor smashed it up and removed the loose protection beyond, so that the weir was only saved from being "gapped" by the presence of the three well-lines. The work as originally constructed consisted of alternate courses

‡ A. N. Khosla, N. K. Bose, and E. M. Taylor, "Design of Weirs on Permeable Foundations." Publication No. 12, Central Board of Irrigation, India. Simla, 1936.

‖ Am. Soc. Mech.E., 1929.

†† "Protection against Scour below River and Canal Works." Minutes of Proceedings, Punjab Engineering Congress, vol. xxvi (1938), p. 107.

of stone in lime-surkhi-sand mortar. On the glacis the upper course had been "blistered" (uplifted) right along the weir, and finally as the 1-foot gap between the wells had not been closed off fully there was every reason to suspect that there were large cavities under the downstream glacis and floor similar to those that actually existed in the semi-pervious floor and down the crest.

The Marala weir was the second most important weir in the Punjab, as upon it depended the irrigation of the Lower Bari Doab colony as well as the Upper Chenab, and neither a dentated sill nor any other arrangements to prevent scour downstream could have saved that weir from ultimate collapse if it had not been reconstructed.

It was true that Dr. Rehbock, late in 1935 and at the request of Mr. E. O. Cox, the Chief Engineer of the Punjab, had conducted a test at Karlsruhe on a model of the existing Marala weir. At the time he was under the impression that the blocks and the loose protection below the last, or A, line of wells (shown in the plan supplied to him) did not exist at the site. In that test the dentated sill was placed at the end of the work above the C line of wells. That test gave very satisfactory results. Owing to the dangers of oblique flow, the Punjab engineers had refused to agree to the removal of the loose protection below the Marala weir to make the dentated sill fully effective, and, in any case, it was doubtful whether that loose protection could have been removed in the case of the Marala weir, as a good deal of it was grouted and consisted of stone to an unknown depth.

The engineers had also refused to adopt a new device (costing, in the case of the Marala weir, Rs. 8/- per foot), which had not been tested out under Indian conditions in the Research Institute at Lahore or elsewhere in India. That was pointed out to Dr. Rehbock, who agreed to tests being made at Lahore with the loose protection in situ and suggested two positions for the sill, one on the C line of wells at the end of the work and the other 44 feet upstream of that line. He also supplied the dimensions of the sill, and in the model-tests at Lahore those dimensions and the positions suggested by Dr. Rehbock had been faithfully adhered to.

The tests showed that the dentated sill was less effective in the model than were the staggered blocks devised by the Research Institute in Lahore. The results of the tests were sent to Dr. Rehbock, but Mr. Cox had received no further communication from him, and presumed that he accepted them as having been correctly carried out.

Dr. Rehbock admitted in 1936 in his communication that, unless his sill was placed at or near the unprotected river bed "it could be of no real use." On the other hand, staggered blocks, which acted on a different principle, had for some years proved to be of considerable value in reducing scour below weirs where loose protection existed beyond those blocks.

Finally, tests on the Rehbock sill had been carried out on only one occasion in the Punjab Irrigation Research Institute, and not for several years, as alleged by Dr. Rehbock.

# ADDITIONAL ORIGINAL COMMUNICATIONS

## RECEIVED BETWEEN THE 1st SEPTEMBER, 1939, AND THE 31st AUGUST, 1940*.

### Titles.



* Available for reference in the Library; includes Papers awaiting publication.

SEWAGE-DISPOSAL.—The Design of Sewage Purification Works. H. C. Whitehead. No. 5235.

SOIL-MECHANICS.—An Apparatus for Measuring the Lateral Pressure of Clay Samples under a Vertical Load. G. M. Binnie and J. A. Price. No. 5242.

STRUCTURES.—Moment Balance: a Self-Checking Method of analysing Rigidly-Jointed Frames. R. J. Cornish. No. 5246.

The Effect of Earthquakes on Framed Buildings. A. J. Ockleston. No. 5241.

Notes on Determining the Stresses in the Guys of Guy-Supported Masts. H. Tooley. No. 5252.

# OBITUARY.

RUSTAT BLAKE[1] was born on the 16th May, 1871, and died in London on the 14th April, 1940. He was educated at Haileybury and Cambridge University, and received his engineering training under Sir John Wolfe Barry and Mr. H. M. Brunel. He remained with the firm of Sir John Wolfe Barry and Partners, and was engaged in railway and constructional work in the United Kingdom and in India. From 1911 to 1918 he was personal assistant and later chief assistant to Sir John, and was responsible for the whole of the firm's work as consulting engineers for the Bengal-Nagpur Railway. He was also joint consulting engineer for the Southern Punjab Railway until its liquidation. During the great war he was a member of the Headquarters staff of the Department of Explosives Supply, Ministry of Munitions. In 1918 he became a partner in the firm of Sir John Wolfe Barry, Lyster, and Partners, and in 1920 was appointed civil engineering member of the Royal Commission on Uniform Railway Gauge for the Commonwealth of Australia, which prepared a report to the Governor-General. In 1922 he joined, as a partner, the firm of Sir Alexander Gibb and Partners, and was actively engaged, until his death, in all branches of constructional engineering, and particularly in connexion with hydro-electric power, tunnelling, waterworks, structural steelwork, docks and harbours, and foundation work. He was a member of the Home Office panel of civil engineers under the Reservoirs (Safety Provisions) Act, 1930.

Mr. Blake was elected an Associate Member of The Institution in February, 1897, and was transferred to the class of Member in March, 1910.

He was also a member of the Institutions of Mechanical Engineers, Locomotive Engineers, and Engineers of Australia, and of the Institute of Metals, and was an Associate of the Institution of Naval Architects.

In 1911 he married Maud, daughter of James Wallace, of Moffat, who survives him.

---

PROFESSOR STEPHEN MITCHEL DIXON, O.B.E., was born in Dublin on the 26th May, 1866, and died at Nice, France, on the 25th March, 1940. He was educated at Rathmines School and Trinity College, Dublin. In 1890 he joined the staff of Lovatt & Company, and worked on the King's Cross and Redhill tunnels. In 1902 he was appointed

[1]. A more detailed memoir of Mr. Blake is available for reference in the Institution Library.

Professor of Civil Engineering in the University of New Brunswick, and after 2 years went to the Dalhousie University, Nova Scotia. In 1905 he became Professor of Civil Engineering at the University of Birmingham, being appointed Dean of the Faculty of Science in 1912. In 1913 he came to London as Professor of Civil Engineering at the Imperial College, South Kensington. He remained in the University of London until 1933, and served as Dean of the City and Guilds College from 1931 to 1933. Early in the great war he was personal assistant to Sir Henry Fowler, the Director of Production at the Ministry of Munitions; in 1917 he went to France as a Lieutenant in the Royal Engineers, and was promoted to Captain in 1918.

From 1923 to 1935 Professor Dixon was a member of the Safety in Mines Research Board, and was in charge of researches on the support of workings. He served as Secretary of the Institution Committee on the Deterioration of Structures exposed to Sea Action, and also carried out important research work. In 1937 he was awarded the O.B.E., and in the same year he retired and took up residence at Nice.

He was elected an Associate Member of The Institution in December, 1892, and was transferred to the class of Member in 1906. He delivered the Vernon Harcourt Lecture in 1929*. He presented Papers to The Institution, sometimes as joint author, on photographic surveying §, on the strength of concrete slabs†, on the flow of water over a broad-crested dam‡, and on the flow of the river Severn||. For the first- and the last-mentioned of these Papers he was awarded Telford premiums.

He married, in 1894, Aline Allison, daughter of Thomas Harrison, Chancellor of the University of New Brunswick, by whom he had one daughter, and who died in 1934; in 1937 he married Mademoiselle Josephine Marie Jud, of Annecy, Haute Savoie, who survives him.

---

* "The Work of the Committee of The Institution on the Deterioration of Structures in Sea-Water." The Inst. C.E., 1930.

§ "Surveying with a Camera." Minutes of Proceedings Inst. C.E., vol. clxxxiv (1910–11, part II), p. 284.

† S. M. Dixon and P. W. Villiers, "Experiments on the Strength of Plain and Reinforced Concrete Slabs." *Ibid.*, vol. clxxxv (1910–11, part III), p. 310.

‡ S. M. Dixon and F. W. Macaulay, "Measurement of Discharge over a Rock-faced Dam." *Ibid.*, vol. 220 (1924–25, part II), p. 98.

|| S. M. Dixon, G. Fitzgibbon, and M. A. Hogan, "The Flow of the River Severn, 1921–36." Journal Inst. C.E., vol. vi (1936–37), p. 81 (June, 1937).

# ABSTRACTS OF THE CURRENT TECHNICAL LITERATURE OF ENGINEERING AND APPLIED SCIENCE.

## ENGINEERING CONSTRUCTION.

**Detection of Unsound Chert in Concrete Aggregate.** C. E. WUERPEL (**Engng. News-Rec.*, 124, *652–654; 9 May 1940*).—The Author describes the equipment used in studies of unsound concrete aggregates containing chert, which demonstrated that the trouble experienced is caused by the presence of pebbles of low specific gravity. It was found that such pebbles can be separated out by flotation in a heavy liquid. The apparatus required and the procedure to be adopted are described in detail.

**A Study of Sieves for Coarse Aggregates.** A. H. D. MARKWICK (**J. Soc. Chem. Ind.*, 59, *88–92; May 1940*).—The Author describes comparative tests made on woven wire and perforated plate sieves ranging in size from $\frac{1}{8}$ inch to $\frac{3}{8}$ inch. Woven wire sieves were found to be considerably less accurate in aperture-size and shape than perforated plate throughout the range of sizes tested. A woven aperture had an effective size larger by about 4 per cent. than that of a perforated plate of equal projected size. The results are presented in Tables and curves, and the Author suggests that woven sieves could be replaced advantageously by perforated sieves down to and including the $\frac{3}{16}$-inch size, where the sieves designated in fractions of an inch join the fine-mesh series and a discontinuity therefore already exists.

**Masonry Dams.** (**Proc. Amer. Soc. Civ. Engrs.*, 66, *812–943; May 1940.*)—In the first of a series of six Papers reviewing progress in connexion with the design and construction of very high dams, Messrs. I. E. Houk and K. B. Keener (*pp. 813–827*) discuss basic assumptions and technical considerations involved in the design of dams of the single-arch, curved gravity, and straight gravity types, built on rock foundations. Mr. R. S. Lieurance (*pp. 829–851*) presents a comprehensive series of Tables to facilitate the computation of forces, moments, and radial deflexions in the design of arch dams. Seven basic load conditions are thus provided for. Messrs. C. H. Paul and Joseph Jacobs (*pp. 852–868*)

NOTES.—An asterisk prefixed to a reference, thus **Engng. News. Rec.*, denotes that the article is illustrated.

The abbreviated titles of periodicals are those used in the "World List of Scientific Periodicals" (Oxford, 1934).

discuss the preparation of rock foundations, including the areas upstream and downstream and out into the abutments adjacent to the limits of the superimposed structure, as far as the loading, seepage, or scour criteria may require. Mr. I. B. Crosby (*pp. 869–890*) discusses the geological problems involved in the economical construction of safe dams. Mr. I. L. Tyler (*pp. 891–907*) reviews the progress in concrete manufacture and control as applied to dams, and discusses future developments. Mr. B. W. Steele (*pp. 908–943*) deals with construction joints, which he defines as formed or unformed horizontal, vertical, or inclined surfaces between masses of concrete deposited at different times.

**Model-Tests of a Bridge-Pier supported on Long Steel Piles.** T. F. Comber, jun., and J. M. Coan, jun. (**Proc. Amer. Soc. Civ. Engrs.*, 66, *1033–1052; June 1940*).—The Authors describe tests made on a model of a proposed deep-water bridge pier. The model conformed to the prototype except for the neglect of the restraining action of the foundation material. The results obtained for five conditions of loading, including direct load, wind load, ice, and combination loading, are tabulated.

**Lengthening a Simple Truss Bridge by Means of a Long Cantilever.** H. J. Engel (**Engng. News-Rec.*, 125, *22–24; 4 July 1940*).—Widening of the Atchafalaya river for flood-control necessitated lengthening of the Missouri Pacific railway-bridge at Klotz Springs, La. For this purpose two of the existing 300-foot simple truss spans were separated by 720 feet of new steelwork in the form of twin cantilevers having a common anchor arm. The 300-foot spans were hung from the ends of the new cantilevers. The work involved the sinking of two deep caisson piers, and the rolling out of one of the simple spans to the far end of the new steelwork, together with the simultaneous rolling in of an equal length of new cantilever steelwork—all without interruption to railway traffic.

**Vertical-Lift Bridge at Cleveland, Ohio.** W. P. Brown (**Civ. Engng., N.Y.*, 10, *429–432; July 1940*).—The straightening and widening of the Cuyahoga river at Cleveland rendered necessary the reconstruction of the Upper West Third Street bridge, and in place of the former swing span a 225-foot vertical-lift span was built. During the reconstruction, in order to provide for the heavy road and river traffic, a by-pass was arranged, including a pontoon swing bridge. The Author describes the design and construction of the foundations, which consisted of cylindrical steel piers, sunk to hard shale about 175 feet below the level of the bridge-floor and filled with concrete. The vertical-lift span is a through-riveted Warren-type truss of twelve panels, 217 feet between centres of bearing. Its depth ranges from 28 feet at the ends to 39 feet at the centre. The trusses are 46 feet apart centre to centre. The operation of the lift span and the precautions adopted to ensure safety are described in detail.

**Tests on Concrete Beams reinforced with High-Tensile Steel and Mild Steel.** G. KAZINCZY (**Concrete Constr. Engng.*, 35, *223–231; May 1940*).—The object of the tests described was to ascertain whether high-tensile steel may be used at its full permissible stress in ordinary-grade concrete. At first the load was gradually increased. When two-thirds of the expected ultimate load was reached the load was removed, and was then applied again. Near the ultimate load, and after it was reached, the load was increased so that the beam was bent uniformly. Measurements were made after every $\frac{1}{25}$ inch of deflexion, and thus it was possible to observe the phenomena in the neighbourhood of failure. The results obtained are tabulated and plotted in curves. The Author concludes that there is no ground for restricting the use of high-tensile steel according to the quality of the concrete; that the load-bearing capacity of reinforced-concrete beams increases in proportion to the yield-point; that the permissible concrete stress, with high-tensile steel, may be increased at least to the extent of the Austrian regulations, and that in all cases wherein the steel is the decisive factor, owing to the small percentage of reinforcement, it is unreasonable to ask for high-quality concrete, since the ordinary run of concrete is usually better than is required.

**Effect of Impact on Reinforced-Concrete Beams.** T. D. MYLREA (**J. Amer. Concrete Inst.*, 11, *581–594; June 1940*).—The Author describes tests made on 10-inch by 16-inch beams on a span of 8 feet, to determine the impact resistance of various quantities of reinforcing steel of different grades. The tests were made first with a hammer weighing 560 lb., and later with a hammer weighing 2,040 lb. The procedure was similar in both series of tests. A light blow—2 inches with the light hammer, or 1 inch with the heavy hammer—was sufficient to cause the concrete to crack. A greater blow would have seriously overstressed the steel. The results are tabulated, and the Author draws the following conclusions: (1) even small quantities of reinforcement may provide a good protection against failure from shock; (2) the factor of safety against rupture of the reinforcement is very high; (3) the effect of yield in the supporting members in buildings, and the cushioning effect of tires and springs on bridges will give an even greater degree of safety; (4) rail-steel and other "brittle" steel reinforcement is quite as resistant to impact as is structural steel reinforcement.

**Reconstruction of the Pigeon House Electricity Generating Station, Dublin.** P. G. MURPHY (**Trans. Instn. Civ. Engrs., Ireland*, 66, *142–169; May 1940*).—The Pigeon House power-station was first put into commission in 1903 with generating plant totalling 3,000 kilowatts. Extensions and replacements were made periodically up to 1929, when the total installed capacity amounted to 21,000 kilowatts. The new reconstruction, completed early in 1940, provided for an ultimate installed

capacity of 100,000 kilowatts. The Author describes the work of reconstruction and the new plant, and indicates how the problems involved in reconstructing a comparatively small power-station to one of five times its output have been solved, utilizing the existing site, buildings, and plant facilities to the greatest possible extent.

**The Security of Vertical Jetties.** LARRAS (**Sci. et Ind. (Travaux)*, 24, *109–114 ; Apr. 1940*).—The Author states that tests on models and on actual structures since 1932 at Algiers enable an appreciation to be made of the risks run by vertical jetties in consequence of errors made in computing the violence of storms, and of the extent to which such risks can be humanly avoided. He discusses the errors which may arise in the evaluation of the characteristic elements of waves, the risks of catastrophe which result therefrom for vertical jetties, and the precautions which should be taken to reduce these risks to the minimum.

**The Bayonne Terminal, Port of New York.** M. GARSAUD (**Civ. Engng., N.Y.*, 10, *425–428 ; July 1940*).—The Author describes the design and construction of an ocean terminal of the quay type, which projects about 2 miles out from the New Jersey shore at Bayonne into the Upper Bay. The special problems involved included the dredging and filling of 8 million cubic yards of material, the construction of a bulkhead about 9,300 feet in length around the working area, and the construction of a connecting causeway about 5,000 feet in length with a top width of 100 feet. The bulkhead was of the relieving platform type and in order to provide for railway tracks and future crane equipment the design was arranged for a live load of 750 lb. per square foot, or a total of 1,500 lb. per square foot. The total cost of the terminal was $4,050,000.

**The Source of Water derived from Wells.** C. V. THEIS (**Civ. Engng., N.Y.*, 10, *277–280 ; May 1940*).—The Author discusses the factors which control the response of water-bearing strata to development by wells, in view of the increasing use of subsoil water for industrial and municipal purposes, and for irrigation. These factors include (1) the distance to, and the character of, the recharge ; (2) the distance to the locality of natural discharge ; (3) the character of the cone of depression in the water-bearing strata. He emphasizes the following points :—(*a*) all water discharged by wells is balanced by a loss of water elsewhere ; (*b*) too great a concentration of pumping in any area should be discouraged. Finally he enumerates the factors governing the development of water-bearing strata from the point of view of the maximum utilization of the supply available.

**Permissible Composition and Concentration of Irrigation Water.** W. P. KELLEY (*Proc. Amer. Soc. Civ. Engrs.*, 66, *607–613 ; Apr. 1940*).—The Author asserts that the permissible salt-content of irrigation water is

limited by variables inherent in the soil, by the climatic conditions, and by the nature of the crops grown. Saline irrigation water must be applied in quantities in excess of the crop requirements, in order that some leaching of the root zone may take place: therefore the maintenance of good drainage of the soil is very important. Salts, whether native to the soil or applied in the irrigation water, cannot be removed effectively unless water can percolate through the soil, and this cannot be accomplished adequately if the ground-water is near the surface.

**Fractionated Coagulation.** R. A. Trelles, D. J. Bengolea, and A. G. Poccard (**J. Amer. Waterw. Ass.*, 32, *742–750; May 1940*).—Fractionated coagulation consists of adding the total dose of coagulant to water in two doses, with an interval between them, and employing either aluminium salts only, or using them for the first dose and iron salts for the second. The Authors describe experiments made upon the Rio Plata supply for the city of Buenos Aires, and present the results obtained. They conclude that fractionated coagulation offers the following advantages:—(1) improvement in quality of the settled water; (2) use of iron salts; (3) more rapid decantation; (4) saving of coagulant.

**Influence of the Rugosity of Pipes and of their Age upon the Hydraulic Delivery.** Roger Mathieu (**Ann. Ponts Chauss.*, 109-ii, *331–396; Oct. 1939*).—The Author discusses the character of the flow in a circular pipe and the practical effect of the nature and condition of the pipe-wall. He reviews the theoretical laws and empirical formulas of flow, and makes a survey of American studies of the influence of the age of a pipe upon the flow through it. Finally, he discusses methods of lining pipes and their influence upon cost and amortization. He concludes that rugosity increases with age and states that the yield may be halved, whilst the resistance or the loss of head may be tripled or quadrupled. This rugosity introduces an element of uncertainty into pipe calculations and imposes the adoption of high margins of safety. With rare exceptions, empirical formulas take no precise account of the effects of age. Recent methodical observations have shown that these effects are often underestimated. Only pipes with smooth walls can be calculated with precision for the duration of their service. This advantage is rendered possible by the use of modern linings, which increase the hydraulic delivery and often enable economics to be realized immediately which are in excess of the cost entailed.

## MECHANICAL ENGINEERING.

**The Circulation of Water and Steam in Water-Tube Boilers, and the Rational Simplification of Boiler Design.** W. Y. Lewis and S. A. Robertson (**J. & Proc. Instn. Mech. Engrs.*, 143, *147–175; June 1940*).—

The Authors discuss the nature of the change from water to steam, discuss the flow in the simple U-tube boiler, and consider the features of various departures from the standard U-tube met with in practice. They describe an improved form of tube, which permits simplification of boiler design, whilst presenting the advantages of good circulation, high gas-speed, and high heat-transmission rates. Further simplifications and improvements are described, which enable substantial reductions to be made in the cost of manufacture, installation, and maintenance of boilers of any pressure and capacity, for service on land or sea.

**A Study of Heat-Insulation Problems in Steam Power Plants.** E. T. Cope and W. F. Kinney (**Mech. Engng., N.Y.*, 62, *465–470 ; June 1940*). —The Authors discuss the insulation of furnace-walls, ducts, and fan-housings, piping and accessories, and steam turbines, and present the following conclusions drawn from a comprehensive survey of heat insulation for turbines, made recently by the Detroit Edison Company :—(*a*) insulating pads, composed of a textile jacket enclosing a loose filler, appear to be the most desirable form of insulation for turbines ; (*b*) for temperatures up to 1,050° F. preference is accorded to a glass-fabric pad with glass-wool filler ; (*c*) 50° F. appears to be a reasonable temperature-difference between the outside surface of the insulation and the ambient air ; (*a*) insulation should be considered for the exposed surface of the admission-valve stem guides on multiple-admission-valve type turbines ; (*e*) in any type of insulation in which the lacing wire is looped around lugs on the turbine casing, heat-resisting wire should be used on those portions of the turbine-casing where the temperature exceeds 850° F. ; wire made from 18–8 stainless steel has proved satisfactory.

**Power-Supply for Central-Station Auxiliaries.** D. B. Reay (**J. S. Afr. Instn. Engrs.*, 38, *350–369 ; May 1940*).—The Author presents a summarized description of the systems of auxiliary supply mainly adopted in modern practice, and describes the methods used in the various power-stations of the Victoria Falls and Transvaal Power Company, Ltd. Operating results are given for these stations, and comparisons are made between the older and the newer stations, in order to illustrate the development of auxiliary power-supply over a period of about 30 years. Finally the Author discusses the protection of the auxiliary equipment—generators, feeders, and motors.

**Anthracite Duff for Power Generation.** (**Power & Works Engr.*, 35, *191–197 ; Aug. 1940.*)—An account is given of the reorganization of the power-supply system for a group of collieries in South Wales, and the construction of a central power-station. The boiler plant—three water-tube boilers, each evaporating 25,000–30,000 lb. per hour at 160 lb. per

square inch and 550° F.—is fired entirely by anthracite duff, utilizing an ingenious arrangement of the combustion-chamber and compartment type chain-grate stokers. The generating plant is described in detail. The equipment also includes a large-scale application of mercury-arc rectifiers, enabling each colliery to obtain its appropriate supply from a common alternating-current source.

**An Investigation of the High-Speed Producer-Gas Engine.** M. W. Woods (**Engineer, Lond.*, 169, *448–450 ; 468–469 ; 17 and 24 May, 1940*).—The Author presents the results of a series of experiments, comprising engine tests on charcoal producer gas at a number of compression-ratios, ranging from 4·91 : 1 to 15·7 : 1, made with the object of analysing the behaviour of producer gas in a high-speed engine. The results are tabulated and plotted in curves, and from them the Author draws the following conclusions :—(1) the producer-gas engine may be run at compression-ratios of up to at least 16 : 1 without detonation or pre-ignition ; (2) with the point of ignition set for maximum power, cylinder-pressures become excessive when the compression-ratio exceeds 15 : 1 ; (3) at all compression-ratios maximum power is attained with a mixture 2 per cent. weaker than the theoretically correct mixture ; (4) owing to the sharply-peaked nature of the power/mixture-strength curve, precise control of mixture-strength is necessary if maximum power is to be developed. In two Appendixes the Author discusses the effect of dissociation upon the combustion of producer gas–air mixtures, and the power output of a petrol engine running on producer gas.

**Experiments on Jerk-Pump Ignition.** K. J. DeJuhasz (**Bull. Penna. State Coll. Engng. Expt. Stn., No. 51, 50 pp. ; 8 Apr. 1940*).—The object of the tests described was to study the influence of pipes of various lengths and diameters, enclosed volumes, and included masses, upon the timing and duration of fuel-injection in diesel engines. The theory of the injection process is discussed, the testing equipment used is described, and the results obtained are presented in numerous diagrams. The Author concludes that the timing (beginning, end, duration) of injection approximates to the pump delivery the more closely (*a*) the shorter the pipe ; (*b*) the smaller the pipe-diameter ; (*c*) the lighter the nozzle-moving parts ; (*d*) the smaller the enclosed volume ; (*e*) the larger the nozzle orifice ; (*f*) the lower the valve-opening pressure. In an Appendix he presents a survey of experimental methods for injection research. A bibliography of 36 references is also included.

**Some Effects of Variable Excitation on Synchronous Motor Oscillation.** J. G. Barry (**J. Franklin Inst.*, 229, *491–511 ; Apr. 1940*).—The experiments described were made with a differential analyser, to study the rotor oscillations resulting from sudden application of mechanical

load to a synchronous motor connected to an infinite bus, and the damping effect of variations in field excitation. It was found that the magnitude and duration of the oscillations could be decreased considerably by using the proper type of excitation variation.

**Electric Motor-Coach Trains.** H. H. ANDREWS (*J. Instn. Loco. Engrs.*, 30, *96–130 ; Mar.–Apr. 1940*).—The Author deals essentially with the electrical characteristics, and touches on coaches and bogies only in so far as these are affected by the electrical equipment. He reviews the development of motor-coach trains since 1894 and gives details of the various types of equipment adopted. The principal characteristics of motor-coaches in service on lines in Great Britain, in Europe, in India, in Australia, and in North and South America are tabulated.

**70-foot Locomotive Turntable for the London Midland and Scottish Railway.** (**Engineering, Lond.*, 149, *621–622 ; 28 June, 1940.*)—The turntable described is capable of turning all classes of engine and tender up to a total weight of 175 tons. It is of the non-balanced or articulated type, the girder on which the rails are carried being provided with a flexible joint, so that it can adapt itself to the load in whatever position the locomotive may be. The rotation of the turntable is effected by the locomotive itself, by the creation of a vacuum on the " exhaust " side of an oscillating engine, and turning through 180 degrees can be effected, without effort on the part of the engine crew, in less than 2 minutes.

**Oxygen Cutting of Steel : Part II—Oxygen Cutting Procedure.** W. SPRARAGEN and G. E. CLAUSSEN (**J. Amer. Weld. Soc.*, 19 (*Weld. Res. Suppl.*), *161–208 ; May 1940*).—The first part of this review of the literature of the subject published up to 1 January, 1939, was dealt with in ENGNG. ABS. (Mech.), 2, No. 360 (*Sept. 1939*).—The second part deals with the cutting procedure, oxygen nozzles, preheating fuels, theories of cutting, and applications of the processes. A bibliography containing 333 references is included, and various research problems are suggested.

**Hard Facing—a Process for the Mechanical Engineer.** E. E. LE VAN (**Mech. Engng., N.Y.*, 62, *459–464 ; June 1940*).—Essentially, hard facing consists of welding on to wearing parts a coating, edge, or point of a hard metal, usually a special alloy possessing high wear-resistance qualities. The Author states that hard-faced parts will outwear plain or unfaced parts from two to twenty-five times, depending upon the type of hard-facing alloy and the service to which the part is subjected. He discusses the requirements and types of the materials required for hard-facing, the preparation of the surfaces to be faced, and the welding methods employed, and describes various applications to tools, coal-mining, railways, power plants, oil-production plants, and internal-combustion engines.

**The Creep Phenomenon in Ropes and Cords.** C. G. Lutts and D. Himmelfarb (**Preprint No. 105, Amer. Soc. Test. Mat., 5 pages; 1940*).—Tests were made in which ⅛-inch diameter cotton ropes and ½-inch diameter manila ropes were loaded under tensile stresses ranging from 41 to 89 per cent. of the breaking strengths of the ropes. The periods of time required for the ropes to break under these static tensions ranged from several minutes at the maximum tension to several hundred hours at the minimum. When first loaded the ropes stretched rapidly, but as the period of tension increased the rate of stretch decreased; approximately 90 per cent. of the total stretch occurred in the same period of time as the last 10 per cent. The Authors conclude that under tension creep continues in a rope until a limiting elongation, represented by the normal elongation at the breaking-point, is reached; this limiting range appears to be constant regardless of the tension in the rope.

**Wire Rope for Construction.** (**Engng. News-Rec.* 124, *734–740; 23 May 1940.*)—Emphasis is laid upon the importance of proper selection and use of wire rope, and the necessity for rope users to familiarize themselves with the various types, qualities, and grades now available. Several types are illustrated, and the grades of steel used are enumerated. The manufacture of wire rope is described in detail. Stresses, factors of safety, and selection problems are discussed.

**The Friction of Lubricated Metals.** F. P. Bowden and L. Leben (**Phil. Trans. Roy. Soc.*, A, 239, *1–27; 27 June, 1940*).—The Authors present an analysis of the kinetic friction between metals sliding under conditions of boundary lubrication. From an investigation of the frictional behaviour of metallic surfaces covered with successive monolayers of lubricant, they conclude that for effective lubrication it is necessary to have present a layer of lubricant several molecules thick, and that boundary lubrication cannot be regarded as a purely surface phenomenon. They develop a theory to explain boundary lubrication, and state that even with lubricated surfaces the local pressures in the region of contact are very high, so that the lubricant film between the surfaces is partly broken down. The frictional behaviour of boundary-lubricated surfaces is largely governed by the extent to which the lubricant film breaks down during sliding.

**Some Experiments on Seizure between Lubricated Hard Steel Balls.** D. Clayton (**J. Inst. Petroleum*, 26, *256–271; May 1940*).—In the tests described, for which the four-ball apparatus was used, the standard conditions were varied by altering the speed, increasing the load, interrupting the seizure by removing the load, and varying the rigidity of the chuck mounting. The results are plotted in curves, and are compared with those obtained by other investigators.

**The Lubrication of Steam-Turbine-Driven Electric Generators.** F. J. Cowlin (**Proc. Instn. Mech. Engrs.*, 143, *83–100 ; May 1940*).—The Author reviews the problems involved, and describes the methods adopted to ensure effective lubrication, including various types of oil-pumps, test results for which are plotted in curves. He outlines the development of modern oil-cooling apparatus and describes a series of tests thereon, reproducing curves showing the relation between heat-transmission rate and oil- and water-velocities. He also discusses the properties of lubricants, factors influencing their deterioration, and methods of purification.

## MINING ENGINEERING.

**Falls in Bord-and-Pillar Workings.** (**Trans. Instn. Min. Engrs.*, 99, *97–111 ; June 1940.*)—The sixth progress report of the North of England Safety in Mines Research Committee describes the results of investigations into the causes of falls in bord-and-pillar broken workings, up to 3 September 1939. The problems dealt with include the nature of the roof loads ; the effects of working in adjacent seams ; loading conditions in broken workings ; the proportion of coal left in whole pillars ; the conservation of the strength of pillars during extraction ; the use of stone-packs in the goaf ; the effect of the direction of the face line in individual lifts ; and roof support in pillar extraction. The results are considered to be of value in suggesting methods of minimizing the roof pressure in broken workings, and thus effecting economies in supporting the roof.

**Coal-Cutting with Mechanical Gummers.** H. W. Smith and G. M. Gullick (**Trans. Instn. Min. Engrs.*, 99, *53–72 ; May 1940*).—The Authors discuss the use of mechanical gummers and present practical results derived from experience. They enumerate the advantages of the device and describe in detail various types of gummer and their operation. They discuss the effect of the use of the gummer with various widths and depths of kerf and present detailed operating results. In an Appendix descriptions are given of tests made with and without mechanical gummers fitted to coal-cutters, and also of experiments on dust and gas-emission. The results of the latter tests are tabulated in detail.

---

Note.—The Institution as a body is not responsible either for the statements made, or for the opinions expressed, in the Papers and Abstracts published.

MADE AND PRINTED IN GREAT BRITAIN BY WILLIAM CLOWES AND SONS, LIMITED, LONDON AND BECCLES.

NOTE.—*Pages* [1] *to* [12] *can be omitted when the Journal is bound in volume form.*

# NOTICES

## No. 8, 1939—40

## OCTOBER, 1940.

## MEETINGS, SESSION 1940–41.

### OPENING MEETING.

The first meeting of the One Hundred and Twenty-Second Session of The Institution will take place on Tuesday, 5 November, at 1.30 p.m., when Sir Leopold H. Savile, K.C.B., will deliver his PRESIDENTIAL ADDRESS.

### ORDINARY MEETINGS.

Arrangements have been made for the following subjects to be brought forward in the first part of the session, on the dates shown below :—

1940.
(1.30 p.m.)

Nov. 19. The Dugald Clerk Lecture. "**Methods of Excavation Work at Home and Abroad,**" by William Barnes, M.I.Mech.E.

Dec. 17. "**Mohammad Aly Barrages,**" by A. G. Vaughan-Lee, M. Inst. C.E. (*Light refreshments will be served at* 12.45 *p.m.*)

## SPECIAL ANNOUNCEMENTS.

### MILITARY SERVICE.

#### ARMY OFFICERS' EMERGENCY RESERVE.

The War Office has informed The Institution that applications for registration in the above-mentioned Reserve are now particularly invited from Corporate Members who are over 31 and not more than 45 years of age, with a view to accepted applicants being subsequently granted Emergency Commissions in the Corps of Royal Engineers. This requirement is therefore brought to the notice of those members who are free and feel disposed to offer their services.

Particulars of the Abridged Conditions of Service, etc., in regard to the Army Officers' Emergency Reserve may be obtained on application to the Secretary of The Institution.

Members who wish to apply for registration should forward their names to the Secretary, who will transmit them to the War Office, in order that a copy of the appropriate form of application (Form E. 564A) may be sent. Completed applications should be returned to The Institution, in order that certificates verifying membership may be attached and the applications then transmitted to the War Office.

Corporate Members over 45 and under 55 years of age may also apply for registration, but the prospect of employment cannot yet be foreseen.

### SPECIAL ENLISTMENT IN THE ROYAL ENGINEERS.

The War Office has notified The Institution that Associate Members and Students of The Institution who are over 23 and under 31 years of age are now eligible for special enlistment in Training Units for the Royal Engineers. In this way they will have special consideration, when they join their Unit, and so have a better chance of obtaining Commissions in the Corps of Royal Engineers than would otherwise be the case. It must be emphasized that the War Office will do no more than arrange for their enlistment and joining and can give no guarantee that such recruits will go subsequently into an Officer Cadet Training Unit, since this must depend entirely upon satisfactory recommendation from their Commanding Officers; but it provides an opportunity of which some Associate Members and Students who are not already serving in H.M. Forces may wish to take advantage. Forms of application may be obtained from the Secretary of The Institution.

### NATIONAL SERVICE (ARMED FORCES) ACT, 1939.

Students of The Institution who are 20 years of age and who are liable for Service under the National Service (Armed Forces) Act, 1939, must register at a Local Employment Exchange when their age-group is called, and may obtain from the Secretary a form of certificate indicating their connexion with The Institution, which, upon production to the Interviewing Officers when their age-groups are called, will, it is anticipated, assist them in being posted to the ranks of the Corps of Royal Engineers or to a technical unit in which their qualifications can be employed.

### AIR MINISTRY.

### ROYAL AIR FORCE VOLUNTEER RESERVE.

The attention of members of The Institution is directed to the fact that the Royal Air Force requires a number of technical officers for employment on engineering and armament duties. Commissions in the Royal Air Force Volunteer Reserve will be granted for the duration of hostilities to suitable applicants between the ages of 21 and 50 years possessing the requisite personal and technical qualifications. The following are the minimum qualifications :—

*Engineer.*

(i) Holders of Mechanical Engineering Degrees, or Civil Engineering Degrees if combined with theoretical knowledge of heat-engines.

(ii) Holders of Mechanical Engineering Certificates, or members of Engineering Institutes who also have 2 years' practical experience.

(iii) Practical mechanical engineers who have served an apprenticeship followed by a number of years' experience in erecting or overhauling internal-combustion engines or aeroplane structures, and with knowledge of the properties of engineering materials.

*Armament.*

(i) Holders of Engineering Degrees or Engineering Certificates, or members of Engineering Institutes with at least 2 years' practical experience, particularly those with experience in armament manufacture.

(ii) Practical engineers who have served an apprenticeship followed by a number of years of practical engineering experience and with knowledge of the properties of engineering materials.

[Students and Corporate Members of The Institution who are desirous of offering themselves as applicants for Commissions as Engineer Officers are requested to note that, according to information received from the Air Ministry, the standard of theoretical heat-engine knowledge required of degree men approaches degree standard and that practical men are required to have expert practical knowledge of internal-combustion engines: as regards knowledge of the properties and testing of engineering materials, this is not expected to be of metallurgical standard.]

The appropriate form of application (No. 1020), and notes on conditions of service, may be obtained from the Secretary of The Institution or from the Air Ministry, S.7.e/5., Adastral House, Kingsway, W.C.2.

The Secretary will be pleased to furnish certificates of membership of The Institution for attachment to applications.

## MINISTRY OF LABOUR.

### SCHEDULE OF RESERVED OCCUPATIONS.

The following entry appears in the Ministry of Labour's Schedule of Reserved Occupations :—

Student engineering apprentice or learner—reserved at and above the age of 18 years.

This entry relates only to a man employed in industry or under articles to a professional engineer who produces a certificate from a university or technical institution or from a professional Institution of Engineers to show that he is within two years of the satisfactory completion of a course of study with a view to offering himself for the first time for :—

(i) an Engineering Degree ;

(ii) an Engineering Higher National Certificate ;

(iii) The Associate Membership Examination of the Institutions of Civil, Mechanical or Electrical Engineers, or the Associate Fellowship of the Royal Aeronautical Society ;

(iv) An engineering examination of similar standing to those in (i), (ii) and (iii) above, e.g. Associate Membership Examinations of the Institutions of Marine, Mining and Structural Engineers, Testamur examination of the Institution of Municipal & County Engineers, Higher Grade Certificate in Gas Engineering.

In so far as The Institution of Civil Engineers is concerned, category (iii) applies to Students who are studying with a view to passing Sections A and B of the Associate Membership Examination within a period of 2 years, and who obtain from the Secretary of The Institution certificates to this effect for production to the Registration Officer of the Local Employment Exchange when they register in their age-groups under the National Service (Armed Forces) Act, 1939.

Before a Student is furnished with a certificate, he must send to the Secretary full details of his present occupation, state the dates when he proposes to sit for Sections A and B of the Associate Membership Examination, and satisfy the Secretary in regard to the steps he is taking to prepare himself to sit for such examination.

---

A Corporate Member who is normally engaged in civil engineering, must register at a Local Employment Exchange when his age-group is called, and should designate himself as a "Civil Engineer" to accord with the Ministry of Labour's "Schedule of Reserved Occupations."

He should obtain beforehand from the Secretary of The Institution a certificate of membership, for production to the Registration Officer.

---

# GENERAL ANNOUNCEMENTS.

## THE JOURNAL.

This Number of the Journal completes Session 1939–40. Members are informed that, owing to restriction in the use of paper and other conditions affecting printing, future issues of the Journal will contain not more than eighty pages. The next Number will be published on the 15th November.

## CHANGES OF ADDRESS.

Owing to the number of changes of address incidental to service with H.M. Forces, it is not practicable to register such addresses in the List of Members. It is therefore suggested that a home (private) address be maintained, from which communications issued by The Institution might be re-directed. If, however, this is impracticable, The Institution may, in special circumstances, arrange for the dispatch of the Journal as issued to a service address.

## SERVICE IN THE FORCES.

**For office purposes, a record is being kept of members' service with H.M. Forces, and members who have not already done so are asked to inform the Secretary of such service, i.e. unit, rank, promotions, decorations, etc.**

## EVACUATION OF CHILDREN OVERSEAS.

During the summer The Institution has been in touch with the Engineering Institute of Canada in regard to a generous offer, sponsored

by that body, to arrange for the children of members of the Institutions of Civil, Mechanical and Electrical Engineers who were being sent to Canada under the Government's Children's Overseas Reception Scheme to be received into the homes of Canadian engineers. The offer was gratefully accepted on behalf of the home Institutions, but, owing to the curtailment of the original Government scheme, it has not been possible to take full advantage of the proffered hospitality. The Canadian engineers have since extended their offer to include children whose parents were able to make private arrangements for their transportation to Canada.

Any members who contemplate sending their children to Canada for the period of the war, either under the Government scheme or privately, and who have no relatives or friends in that country to whom to send them, should communicate with the Secretary Inst. C.E. for further information, without delay.

A similar offer has also been received from the Institution of Engineers, Australia, should means of transport be available, and the Australian Institution is ascertaining from its members the number of children for whom provision could be made. A further announcement will be made when additional particulars are received from the Australian Institution.

## EXAMINATIONS.

After the October, 1940, Examination, the Preliminary Examination will be replaced by the Common Preliminary Examination, which will be conducted by the Engineering Joint Examination Board.

The conditions of entry for this Examination and the latest dates for entry will be the same as for the Preliminary Examination which it replaces.

Further information may be obtained on application to the Secretary.

The following dates have been fixed for the Examinations to be held in 1941.

*Common Preliminary Examination.*
April the 1st to the 4th, inclusive.
October the 7th to the 10th, inclusive.

*Associate Membership Examination.*
April the 21st to the 25th, inclusive.
October the 13th to the 17th, inclusive.

The April, 1941, Examinations will be held in London and the Provinces. Intending candidates are reminded that applications to attend should reach the Secretary's hands by the 28th February and that Students of The Institution entering for the Associate Membership Examination are recommended to lodge their applications a fortnight before that date.

## C.C. LINDSAY CIVIL ENGINEERING SCHOLARSHIPS.

Regulations for the award of these Scholarships, sanctioned by the Board of Education, may be obtained on application to the Secretary of the Glasgow and District Association (Mr. William MacGregor, B.Sc., Engineering Department, The University, Glasgow, W.2). Eligibility for the award of these Scholarships, which are each of the value of not less than £25 per annum, is confined to Students of The Institution who are members of the Glasgow and District Association of The Institution and who are British subjects of Scottish parentage.

## HONORARY MEMBERS.

The Council, being of opinion that it is not in the interest of The Institution that the present King of Italy and the present King of the Belgians should remain Honorary Members, acting under the provisions of the Royal Charter, have ordered the removal of these names from the Institution Roll.

## AWARDS FOR PAPERS.

A list of the awards made by the Council for Papers presented in Session 1939–40 appears at pp. 478 and 479, *post*.

## CHARLES HAWKSLEY PRIZE.

On the report of the judges, the Council has awarded a Charles Hawksley Prize of £150 for 1940 to William Eugene Blackmore, Stud. Inst. C.E., of Colchester, for his design of a water tower.

Having regard to the merit displayed by Arthur Richard Collins, M.Sc., Assoc. M. Inst. C.E., and by James Kenneth McIntyre, Stud. Inst. C.E., in their designs of a traffic roundabout and underground garage, the Council has decided that they be "honourably mentioned," and that grants of £30 and £20 respectively be awarded to them in recognition of their good work, on the understanding, however, that these latter grants do not entitle the recipients to call themselves "Charles Hawksley Prizemen."

In view of the large numbers of Students and younger Associate Members who are expected to be serving with H.M. Forces, the Council has decided not to hold a competition for the award of the Charles Hawksley Prize in 1941.

## BAYLISS PRIZE.

On the results of the April, 1940, Associate Membership Examination (Sections A and B) at home and abroad, the Council have awarded a Bayliss Prize of £15 to each of the following Students of The Institution,

Alfred Donald Alsop of Cheadle, Cheshire,

and

Sekaripuram Ramaswamy Krishnaswamy Iyer of Colombo, Ceylon,

whose performances in the examination were of equal merit and who obtained the highest marks of those candidates who fulfil the other conditions for the award of the Bayliss Prize.

## INDEX TO JOURNAL.

For the convenience of members and others who bind the eight numbers of the Journal in volumes, an extra copy of the combined index for the eight numbers will be supplied with the two binding cases, in addition to the index which appears in this Number for binding in the last volume for the year.

## BINDING CASES FOR THE JOURNAL.

Members are reminded that binding cases for the eight numbers of the Journal (November, 1939, to October, 1940) for Session 1939–40 will be available soon after the issue of this Number. Details of charges, etc., will be found in the "Notices" Section of the November, 1939, Journal.

## THE LONDON SOCIETY.

The Council have appointed Sir George W. Humphreys, K.B.E. (Past-President), as representative of The Institution on the Council of the London Society, in succession to the late Mr. T. H. Bailey.

## UTILIZATION OF OLD TRACING LINEN.

The Women's Volunteer Service Headquarters point out that old linen drawings and tracings can easily be utilized for the making of all kinds of articles such as surgical war stores, where the use of fine linen is necessary, and request that all available tracing linen should be collected for use.

It is asked that old tracings, etc., should be addressed to Mr. F. R. Yerbury, Director, The Building Centre, 158 New Bond Street, London, W.1.

## TRANSFERS, ELECTIONS, AND ADMISSIONS.

Since the 7th May, 1940, the following elections have taken place:

| *Meeting.* | *Associate Members.* |
|---|---|
| 11 June. | 83 |

and during the same period the Council have transferred twelve Associate Members to the class of Members, and have admitted one hundred and fifty-nine Students.

## DEATHS AND RESIGNATIONS.

The Council have received, with regret, intimation of the following deaths and resignations:—

DEATHS.

THOMSON, *Sir* Joseph John, O.M., D.Sc., LL.D., F.R.S. (E. 1924.) *Honorary Member.*
ANDERSON, George. (E. 1880.) *Member.*
ANSTEY, *Engr.-Comdr.* Henry Charles, O.B.E., R.N. (*ret.*) (E. 1905. T. 1908.) ”
BABTIE, John Taylor. (E. 1894. T. 1907.) ”
BEARE, *Professor Sir* Thomas Hudson, LL.D., B.A., B.Sc., F.R.S.E. (E. 1885. T. 1893.) ”
BODY, John Benjamin. (E. 1892. T. 1901.) ”
BRICKNELL, Frederick William. (E. 1886. T. 1916.) ”
CASE, Albert Havelock. (E. 1884. T. 1893.) ”

| | |
|---|---|
| FELKIN, Leonard George. (E. 1900. T. 1916.) | *Member.* |
| GAIRNS, Robert Wylie. (E. 1908. T. 1928.) | " |
| GRAHAM, William Vaux. (E. 1896.) | " |
| HAMER, William Henry. (E. 1895. T. 1905.) | " |
| JENKIN, *Professor* Charles Frewen, C.B.E., LL.D., M.A., F.R.S. (E. 1891. T. 1909.) | " |
| MCCLURE, Hugh Hannay. (E. 1902.) | " |
| MACDONALD, Arthur Cameron, D.S.O. (E. 1902. T. 1916.) | " |
| MASSEY, William Henry, M.V.O. (E. 1888.) | " |
| NICHOLSON, Walter Sery. (E. 1881. T. 1908.) | " |
| SHARROCK, Francis Gilbert. (E. 1906.) | " |
| SIBBERING, George Thomas. (E. 1887. T. 1896.) | " |
| SIKES, Robert Cherry, B.A., B.E. (E. 1902. T. 1905.) | " |
| THOMAS, John Frederick Ivor, O.B.E. (E. 1896. T. 1918.) | " |
| WEBSTER, Henry, B.A.I. (E. 1892. T. 1894.) | " |
| ANGLESEA, William George. (E. 1910.) | *Associate Member.* |
| BARRATT, Samuel Harry Hill. (E. 1894.) | " " |
| BAZAZ, Ganesh Dass, B.Sc. (E. 1920.) | " " |
| BLACKADDER, *Professor* William, D.Sc. (E. 1907.) | " " |
| CURRY, William Elmitt. (E. 1883.) | " " |
| *DUDLEY, Kenneth Dening, B.Sc. (E. 1939.) | " " |
| GRAY, David, B.E. (E. 1934.) | " " |
| *HALES, Casper Henry. (E. 1928.) | " " |
| *HAVILAND, Richard Haviland, B.Sc. (E. 1940.) | " " |
| *HAXELL, Charles Ivan. (E. 1938.) | " " |
| MOLLOY, Frederico George, B.Sc. (E. 1927.) | " " |
| PERCY, Norman Crook. (E. 1908.) | " " |
| PORTEOUS, Norman, D.S.O., M.C. (E. 1908.) | " " |
| POWNALL, John. (E. 1894.) | " " |
| SANDEMAN, Harold Fitzroy, B.Sc. (E. 1933.) | " " |
| SMITH, William Alfred. (E. 1920.) | " " |
| TIMMINS, Arthur. (E. 1889.) | " " |
| TONGE, John Henry. (E. 1897.) | " " |
| *VENABLES-LLEWELYN, George William Dillwyn. (E. 1937.) | " " |
| WARREN, Charles Henry. (E. 1915.) | " " |
| WHITEHEAD, Cecil. (E. 1938.) | " " |
| WRIGHT-NOOTH, Rodney George, M.C. (E. 1916.) | " " |
| *HYETT, David Ross. (A. 1939.) | *Student.* |
| *JAMES, Geoffrey Halsted. (A. 1937.) | " |
| *LONGWORTH, William Booth. (A. 1937.) | " |

* Killed on active service.

## RESIGNATIONS.

| | |
|---|---|
| ABELL, *Sir* Westcott Stile, K.B.E., M.Eng. (E. 1915.) | *Member.* |
| THOMAS, Herbert Percival, B.Sc. (E. 1934.) | " |
| BLAKE, George Robertson. (E. 1897.) | *Associate Member.* |
| BREWER, Robert Wellesley Antony. (E. 1901.) | " " |
| BURDETT, Henry Leatham, O.B.E. (E. 1925.) | " " |
| BUTLER, David Butler. (E. 1895.) | " " |
| BUTLER, Matthew Arnold. (E. 1924.) | " " |
| DARWIN, Errol Raffael Henry, B.Sc. (E. 1923.) | " " |
| FERGUSON, Andrew Bainbridge. (E. 1911.) | " " |

| | |
|---|---|
| Stelfox, Sydney Herbert, B.Sc. (E. 1904.) | *Associate Member* |
| Stubbs, Herbert Edward Crampton. (E. 1899.) | " " |
| Taylor, Thomas Harold. (E. 1908.) | " " |
| Ware, Charles Coburn, B.C.E. (E. 1922.) | " " |
| Wonham, Frederick William. (E. 1916.) | " " |
| Beaver, Hugh Eyre Campbell. (E. 1932.)—elected Member. | *Associate.* |
| Eades, Wilfrid George. (A. 1938.) | *Student.* |

## HONOURS.

The Council have much pleasure in congratulating the following Members and Students on the Distinctions conferred upon them :—

*O.B.E.*

Humphreys, Col. Cecil Lee Howard, T.D., R. Signals. *Member of Council.*

"For distinguished services in the field."

*D.S.O.*

Kennedy, Lt.-Col. John Ross, B.Sc., R.E. *Associate Member.*

"For gallant and distinguished services in action in connexion with recent operations."

*M.C.*

Collins, 2nd Lieut. (Acting Lt.) John Jerome, R. Signals. *Student.*

"This officer found himself at La Panne on the beach without any proper signals staff. He, however, not only ran a corps signal office for some time, using dispatch riders and orderlies, without a relief, but personally found and laid cables with his own hands under shell fire and bombing. He was consistently cool in a situation of great difficulty and set a fine example to the men under him."

Hubbard, Lieut. Bruce Lancelot Fortescue, B.Sc., R.E. *Student.*

"For meritorious service in connexion with the War."

Smith, Lieut. David Arthur, R.E. *Student.*

"This officer was in charge of three detachments of The Royal Monmouthshire Royal Engineers, who throughout the period carried out invaluable work to delay the enemy's advance. They never failed to accomplish any task which they were set. On May 11 and 12 he ensured the demolition of bridges over the River Ghette which had been prepared by the Belgian Army. On May 14, in spite of enemy bombing, he completed the destruction of bridges over the River Dendre, north of Louvain. At Ypres on May 26, in spite of continuous shelling and bombing, working day and night with a handful of men, he prepared and fired demolitions in front of the Menin Gate. He yet again made the demolitions a certainty over the River Yser and without his immediate aid many bridges, important to the enemy, would have remained intact."

---

# LOCAL ASSOCIATIONS.

*North-Western Association.*

The Committee has decided that meetings of the North-Western Association be deferred for the time being.

# RECENT ADDITIONS TO THE LIBRARY.

[Journals, Proceedings of Societies, etc., are not included.]

AERIAL PHOTOGRAPHY. *See* SURVEYING.

AERO-ENGINES. CAULFIELD, O. "Aero-Engines for Pilots and Ground Engineers." 1940. Pitman. 5*s*.

—— JUDGE, A. W. "Aircraft Engines." Vol. 1. 1940. Chapman & Hall. 16*s*.

AIRCRAFT. CROOK, W. E. "Electricity in Aircraft." 1940. Pitman. 5*s*.

—— WYBROW, S. G. "Electrical and Wireless Equipment of Aircraft." 4th ed. 1940. Pitman. 5*s*.

AIR-DEFENCE. MINISTRY OF HOME SECURITY. "Choice and Adaptation of Shelters in Houses." 1940. 3*d*.

—— —— "Protection of Windows." A.R.P. Memo. No. 12. 1940. H.M.S.O. 4*d*.

—— —— "Fire Protection and A.R.P. Year Book, 1940–41." 1940. Lomax, Erskine. 7*s*. 6*d*.

AUTOMOBILE ENGINES. EVERETT, H. A., and KELLER, G. H. "Service Performance of 8 Lubricating Oils in Automobile Engines." Eng. Exp. Stn. Bulletin No. 50, Pennsylvania State College. 1940. 50 cents.

BIOGRAPHY. HALÉVY, D. "Vauban: Builder of Fortresses." 1924. Bles. 6*s*.

BOILERS. WALLIS, R. P. "High Pressure and Forced Circulation Boilers." 1940. Electrical Power Engineers' Association, 102 St. George's Square, S.W.1. 1*s*.
*See also* CHEMISTRY.

BUILDING MATERIALS. *See* FIRE AND FIRE PREVENTION.

CHEMISTRY. FRANCIS, W. "Boiler House and Power Station Chemistry." 1940. Arnold. 15*s*.

CHIMNEYS. TAYLOR, C. P., and TURNER, L. "Reinforced Concrete Chimneys." 1940. Concrete Publications. 8*s*. 6*d*.

COMPRESSED AIR. AMERICAN SOCIETY OF MECHANICAL ENGINEERS. "Safety Code for Compressed-Air Machinery and Equipment." 1939. The Society. 30 cents.

DAMPNESS. DAWES, S. B. "Dampness in Domestic Buildings." 1939. Technical Press. 2*s*. 6*d*.

DIVING. ELLSBERG, E. "Men under the Sea." 1940. Harrap. 12*s*. 6*d*.

ECONOMICS. BURNHAM, T. H. "Engineering Economics, Book II." 5th ed. 1940. Pitman. 10*s*. 6*d*.

—— FAY, C. R. "English Economic History, mainly since 1700." 1940. Heffer. 5*s*.

ELECTRICAL-MACHINERY. BULL, H. S. "Direct Current Machinery." 1939. Chapman and Hall. 15*s*.

ELECTRICITY. ADMIRALTY. "Naval Electrical Manual, Vol. 1." 1940. H.M.S.O. 12*s*. 6*d*.

ELECTRICITY-DISTRIBUTION. BECKETT, C. S. "Generation and Transmission." 1940. Blackie. 5*s*.

—— SANDFORD, F. "Electric Distribution Fundamentals." 1940. McGraw-Hill. 17*s*. 6*d*.

ELECTRONICS. ALBERT, A. L. "Fundamental Electronics and Vacuum Tubes." 1938. Macmillan Co. 20*s*.

EMULSIONS. *See* OIL EMULSIONS: ROADS AND STREETS.

EROSION. LITTLE, J. M. "Erosional Topography and Erosion." 1940. A. Carlisle & Co., San Francisco.
*See also* SOILS.

FIRE AND FIRE PREVENTION. "Fire Protection and A.R.P. Year Book, 1940–41." 1940. Lomax, Erskine. 7*s.* 6*d.*

—— FOSTER, H. D. "A Study of the Fire Resistance of Building Materials." 1940. Ohio State University, Eng. Exp. Stn. Bulletin. No. 104. 60 cents.

FOUNDRIES. LAING, J., and ROLFE, R. T. "Non-Ferrous Foundry Practice." 1940. Chapman and Hall. 21*s.*

GASES. KENNARD, E. H. "Kinetic Theory of Gases." 1938. McGraw-Hill. 33*s.*

GEOMETRY. ABBOTT, W. "Practical Geometry and Engineering Graphics." 3rd ed. 1939. Blackie. 11*s.*

HOISTING MACHINERY. ATHERTON, W. H. "Hoisting Machinery." 1940. Technical Press. 32*s.* 6*d.*

HOROLOGY. HOPE-JONES, F. "Electrical Timekeeping." 1940. N.A.G. Press. 10*s.* 6*d.*

HYDRO-ELECTRIC POWER STATIONS. CRELLIN, E. A. "Hydro-electric Power Stations." 1935. International Textbook Co. 8*s.*

INDUSTRIAL REPORTS. NELSON, J. R. "Writing the Technical Report." 1940. McGraw-Hill. $2.50.

—— RAUTENSTRAUCH, W. "Industrial Surveys and Reports. 1940. Chapman and Hall. 16*s.* 3*d.*

KINEMATICS. *See* MACHINERY.

LAW AND LEGISLATION. CRESWELL, W. T., and GREIG, N. P. "Law relating to Building and Engineering Contracts." 3rd ed. 1940. Pitman. 16*s.*

—— SADLER, W. C. "Legal Aspects of Engineering." 1940. Chapman and Hall. 26*s.*

LUBRICATION. *See* AUTOMOBILE ENGINES.

MACHINERY. BROWN & SHARPE MFG. CO. "The B. & S. Handbook. A Guide for young Machinists." 1939. Buck and Hickman. 75 cents.

—— COLVIN, F. H., and STANLEY, F. A. "American Machinists' Handbook. 7th ed. 1940. McGraw-Hill. 26*s.*

—— STEEDS, W. "Mechanism and the Kinematics of Machines." 1940. Chapman and Hall. 18*s.*

—— TUPLIN, W. A. "Vibration in Machinery." 1940. Pitman. 7*s.* 6*d.*

MATERIALS. RADER, L. F. "Materials of Construction." 5th ed. 1939. Chapman and Hall. 24*s.*

MATHEMATICS. KÁRMÁN, T. V. and BIOT, M. A. "Mathematical Methods in Engineering." 1940. McGraw-Hill. 26*s.*

—— WARREN, A. G. "Mathematics applied to Electrical Engineering." 1939. McGraw-Hill. 17*s.* 6*d.*

METALLURGY. HEYER, R. H. "Engineering Physical Metallurgy." 1940. Chapman and Hall. 25*s.*

NAVAL ARCHITECTURE. ROSSELL, H. E., and CHAPMAN, L. B. "Principles of Naval Architecture." 2 vols. 1939. American Society of Naval Engineers. 12 dollars.

OIL EMULSIONS. SHEA, G. B. "Preventing and Treating Crude-Oil Emulsions." 1939. U.S. Bur. Mines Bulletin 417. Supt. of Documents, Washington. 60 cents.

OIL-SHALE. INSTITUTE OF PETROLEUM. "Oil-Shale and Cannel Coal. Proceedings of Conference, 1938." 1938. The Institute. 30*s.*

PILES AND PILING. BRITISH STEEL PILING CO. "The B.S.P. Handbook." 5th ed. 1940. The Company. No price.

POWER STATIONS. *See* CHEMISTRY.

RAILWAYS. PARMELEE, J. H. "The Modern Railway." 1940. Longmans, Green. 25*s.*

—— "Railway Engineering and Maintenance Cyclopedia." 4th ed. 1939. Simmons, Boardman. 5 dollars.

—— "Universal Directory of Railway Officials and Railway Year Book, 1940–41." 1940. Directory Publishing Co. 20*s.*

REPORT-WRITING. *See* INDUSTRIAL REPORTS.

ROADS AND STREETS. GARNER, F. H., and others. "Modern Road Emulsions." 2nd ed. 1939. 11 Bow Church Yard, E.C.4. 10*s.*

—— *Roads and Road Engineering* "Year Book and Directory, 1940–41." 1940. Carriers Publishing Co. 10*s.* 6*d.*

SANITATION. HOPKINS, E. S., *Ed.* "Elements of Sanitation" 1940. Chapman and Hall. 21*s.*

SHEET-METAL. DYER, H. J. "How to Work Sheet Metal." 1940. P. Marshall. 2*s* 6*d.*

—— NEUBECKER, W. "Sheet-Metal Work." 1939. American Technical Society, Chicago. 15*s.*

SHIPS. BRITISH CORPORATION REGISTER OF SHIPPING AND AIRCRAFT. "Rules for the Construction and Classification of Steel Ships and their Machinery." 1939. Glasgow. 15*s.*

—— TALBOT-BOOTH, L. C., *Ed.* "Merchant Ships, 1940." Sampson, Low. 52*s.* 6*d.*

SOILS. BENNETT, H. H. "Soil Conservation." 1939. McGraw-Hill. 40*s.*

SOUND. FITZMAURICE, R., and ALLEN, W. "Sound Transmission in Buildings." 1939. H.M.S.O. 4*s.*

STEEL. SIMONS, E. N., and GREGORY, E. "Steel Manufacture Simply Explained." 1940. Pitman. 6*s.*

STRENGTH OF MATERIALS. MORLEY, A. "Strength of Materials." 9th ed. 1940. Longmans, Green. 15*s.*

SURVEYING. HART, C. A. "Air Photography applied to Surveying." 1940. Longmans, Green. 25*s.*

TECHNICAL REPORTS. *See* INDUSTRIAL REPORTS.

TELEVISION. MOSELEY, S. A., and BARTON-CHAPPLE, H. J. "Television." 5th ed. 1940. Pitman. 10*s.* 6*d.*

TERMITES. NIGERIA PUBLIC WORKS DEPARTMENT. "Notes on Termites and Anti-Termite Work." 1935. Government Printer, Kaduna. No price.

TOWN PLANNING. SHARP, T. "Town Planning." Penguin Books. 6*d.*

—— THOMSON, T. F., and CUMBERBIRCH, A. "Town and Country Planning Act, 1932. Procedure and Progress Schedule." 1939. Alden Press, Oxford. 5*s.*

TRANSJORDAN. IONIDES, M. G. "Water Resources of Transjordan." 1940. Crown Agents. 30*s.*

VACUUM-TUBES. ALBERT, A. L. "Fundamental Electronics of Vacuum Tubes." 1938. Macmillan Co. 20*s.*

VIBRATION. *See* MACHINERY.

WATER-RESOURCES. *See* TRANSJORDAN.

WELDING. LINCOLN ELECTRIC CO. "Procedure Handbook of Arc Welding Design and Practice." 6th ed. 1940. The Company, Welwyn Garden City. 9*s.* 6*d.*

WIRELESS. CAMM, F. J. "Practical Wireless Service Manual." 2nd ed. 1940. Newnes. 6*s.*

# COMBINED NAME-INDEX

AND

# COMBINED SUBJECT-INDEX

FOR

Volumes 13 and 14, 1939–40

Journals, November 1939, to October 1940

# COMBINED NAME-INDEX

FOR

Volumes 13 and 14, 1939–40.

November 1939, to October 1940, Journals.

**13** refers to Volume 13, 1939–40, containing November and December, 1939, and January and February, 1940, Journals.
**14** refers to Volume 14, 1939–40, containing March, April, June, and October, 1940, Journals.

# COMBINED SUBJECT-INDEX

## FOR

## Volumes 13 and 14, 1939–40.

### November 1939. to October 1940 Journals.

---

**13** refers to Volume 13, 1939–40, containing November and December, 1939, and January and February, 1940, Journals.
**14** refers to Volume 14, 1939–40, containing March, April, June, and October, 1940, Journals.

---



---

[1] References in bold type are to Papers published by The Institution.
[2] References in ordinary type are to abstracts from the Technical Literature.

---

MADE AND PRINTED IN GREAT BRITAIN BY WILLIAM CLOWES AND SONS, LIMITED, LONDON AND BECCLES.